STRUCTURAL ALTERNATIVES FOR TDG ABATEMENT AT GRAND COULEE DAM

FEASIBILITY DESIGN REPORT

U. S. DEPARTMENT OF THE INTERIOR
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October, 2000

Prepared for
U. S. Bureau of Reclamation
Pacific Northwest Region

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Executive Summary

Total dissolved gas (TDG) levels in the Columbia River downstream from Grand Coulee Dam commonly exceed the Washington State And Colville Confederated Tribes' water quality standard of 110 percent of saturation. Exceedances are due to the combined impacts of spill operations at Grand Coulee Dam and the downstream transfer of flow with high levels of TDG generated at upstream dams. Concerns regarding Columbia River dissolved gas supersaturation problems and potential for gas bubble disease and mortality in anadromous fish are receiving increased attention because of the Endangered Species Act (ESA) salmon recovery efforts through facilities on the mid- and lower-Columbia and Snake Rivers.

Reclamation is aware of the concerns of regional fish managers and water quality management agencies regarding potential for damage to aquatic resources downstream from the project and has been working within the National Marine Fisheries Service (NMFS) regional forum to achieve short- and long-term resolutions of the problem. Resolutions include investigation of possible structural modifications to Grand Coulee Dam for gas abatement during outlet works releases. In addition, a system study involving the U.S. Army Corp of Engineers (USACOE) Chief Joseph Dam and Reclamation's Grand Coulee Dam has looked at combined effects of structural modifications at Chief Joseph and operational changes involving both structures.

Results from this study indicate that the ability to reach 110 percent TDG in the river below Grand Coulee is more dependent on the TDG levels present in the reservoir than on any of the structural or operational changes studied. A 110 percent saturation level is only attainable for combined spill and power releases if the initial TDG saturation level of Franklin Delano Roosevelt Lake is at or below 105 percent and with relatively high flow in the river.

Feasibility investigations for the proposed structural modifications to Grand Coulee Dam for TDG abatement were performed at Reclamation's Technical Service Center in Denver, Colorado. The previous concept study yielded three structural modification alternatives for further study. The following three alternatives were investigated:

- Mid-level outlets extended and covered for submerged release
- Deflectors positioned below the mid-level outlets
- Forebay pipe with cascade

Each of the three alternatives was studied in a physical hydraulic model at the U. S. Bureau of Reclamation's (Reclamation's) Water Resources Research Laboratory in Denver, Colorado. In addition, structural design and cost estimating information was provided by engineers from the Waterways and Concrete Dams, the Hydraulic Equipment, and the Estimating, Specification, and Value Program Groups.

Estimates of the TDG production were determined for each alternative based upon the 7-day, 10-year flood event with a spill of 50,000 ft^3/s and a total flow of 241,000 ft^3/s. The estimates should be considered conservative due to possible surface off-gassing that could occur in the prototype that was not considered in the estimates of TDG from model observations. The estimated TDG produced by each alternative for the design flow was then mixed with the TDG level of the powerplant releases, for a range of powerplant releases, to predict total mixed river TDG saturation levels. The mixed TDG level is evaluated and recorded at the fixed monitoring station located downstream from the dam.

The TDG production of spill from the existing outlet works is estimated at 190 percent. Mixing this highly supersaturated outlet works spill flow with powerplant releases of varying TDG levels will result in various TDG saturation levels in the river downstream from Grand Coulee Dam, figure E1. Each structural modification was compared to these base existing conditions.

Extending and covering eight of the mid-level outlets to provide submerged releases to the flip bucket resulted in vertical mixing and an estimated generated TDG level of 119 percent. Compared to the existing outlet works release conditions, this alternative is expected to decrease TDG saturation levels in the river by about 14.7 percent. However, when reservoir TDG levels exceed 119 percent the alternative will transfer reservoir TDG levels. The estimated total cost for modifying 8 outlet pairs for the extend and cover alternative is $96.2 million.

Six deflectors below the mid-level outlets at El. 965 produce skimming flow in an attempt to reduce TDG production. TDG performance was highly variable due to wide fluctuations in tailwater elevations. This alternative is expected to produce a TDG level of 128 percent for the design spill conditions. Compared to the existing outlet works releases, this alternative would reduce river TDG levels by about 12.9 percent. One aspect of this alternative not investigated due to physical model limitations, is the impact of this alternative on the river bank stabilization downstream of Grand Coulee Dam. The cost estimate should therefore be considered to be the cost for the structural work on the dam only, and may increase due to costs associated with remedial work for the river bank stabilization. The estimated total cost for modifying 6 outlet pairs for the deflector alternative is $15.6 million.

A pipe extending from the existing forebay dam of the Third Powerplant to a manifolded gate chamber, stilling well, and finally over a baffled cascade attempts to reduce TDG levels by providing active stripping and reducing plunge into the tailwater pool. This third alternative is estimated to produce a TDG saturation level in the spill volume of 125 percent. Compared to the existing outlet works releases, this alternative would decrease river TDG levels by about 13.5 percent. The estimated total cost for the forebay pipe with cascade alternative is $437 million.

Figures E1, E2, E3, and E4 compare the TDG performance of the existing outlet works and each of the three structural alternatives at chosen reservoir TDG levels of 110, 125, and 130 percent for the designed spill release and various total river flows.

The TDG benefit of joint operation with Chief Joseph Dam would be reduction of the spill release at Grand Coulee Dam. With increased powerplant production transferred to Grand Coulee from Chief Joseph, it is estimated that the TDG production could be reduced with no immediate capital investment at Grand Coulee. Levels of 5 to 7 percent reduction in TDG (over the existing outlet works performance) could be realized by a reduction in outlet works release of 20,000 ft^3/s. However, when compared with any of the three structural alternatives, additional reductions of only 1 to 2 percent are noted for initial TDG levels of up to 125 percent in the reservoir.

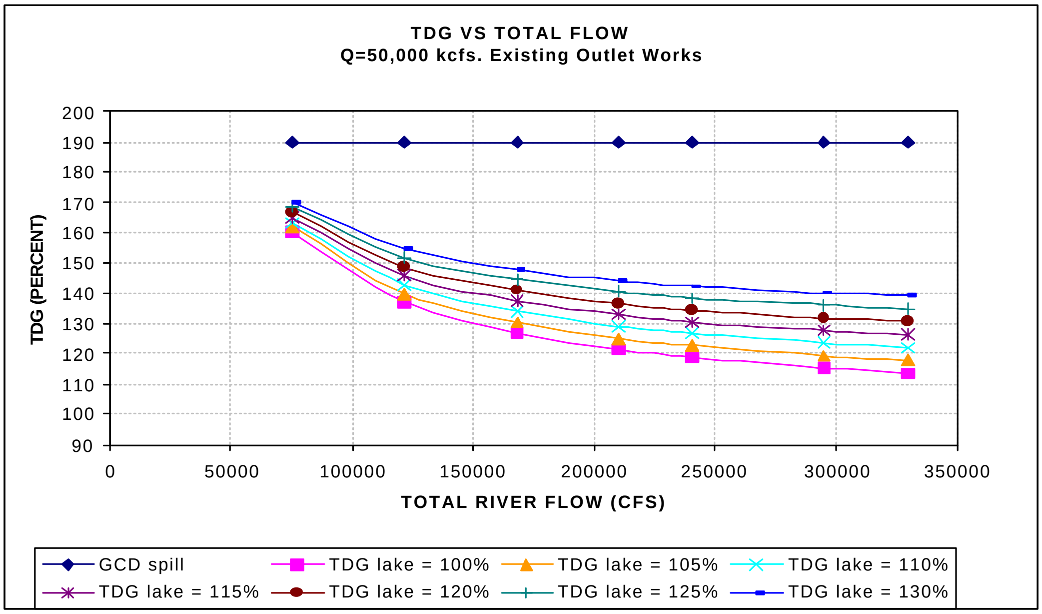


Figure E1. - Existing outlet works TDG production with the 50,000 ft^3/s design discharge passed and then mixed with various lake TDG levels through powerplant releases. (Identical to figure 5)

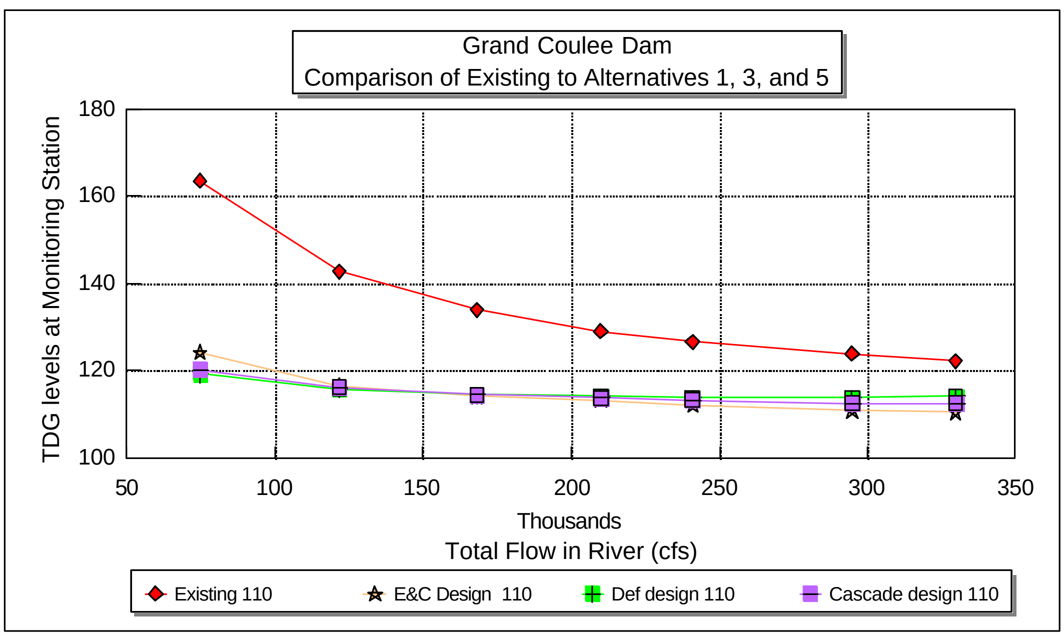
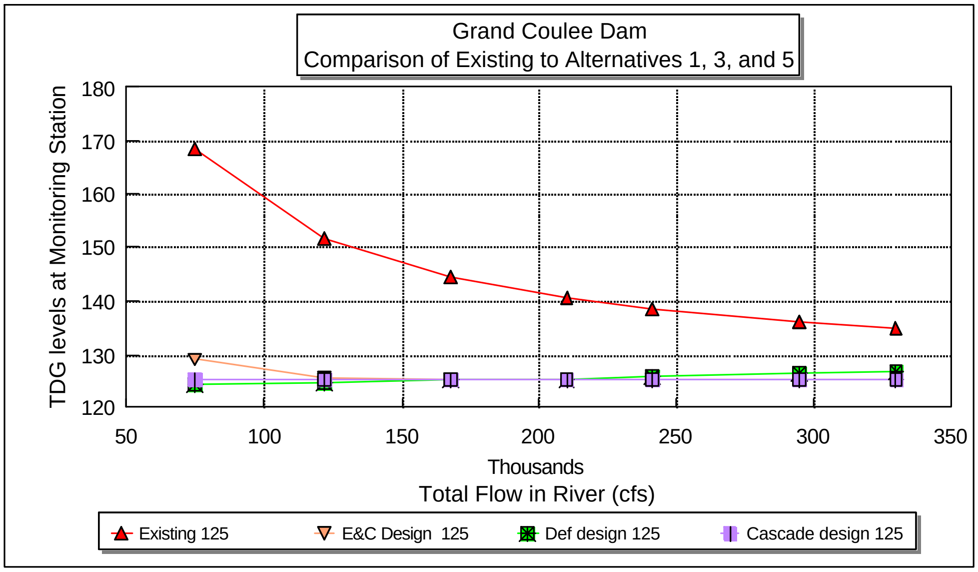


Figure E2. - Comparison of the existing outlet works and alternatives for reservoir TDG level of 110 percent and various total river flows. (Identical to figure 53)



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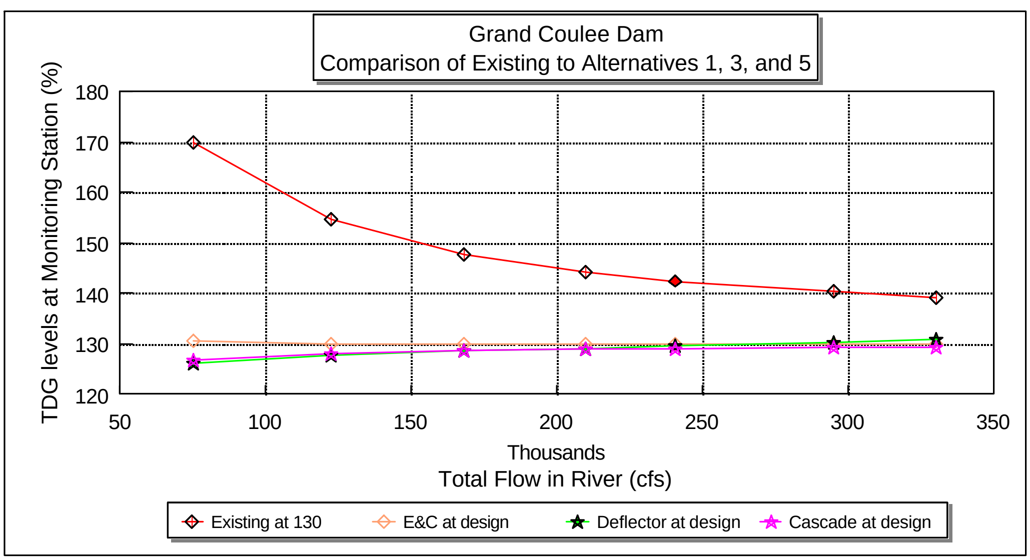

Figure E4. - Comparison of the existing outlet works and alternatives for reservoir TDG level of 130 percent and various total river flows. (Identical to figure 55)

Acknowledgments

The information compiled in this report was prepared with the assistance of the Technical Service Center Grand Coulee Dam Total Dissolved Gas Team. The team included Kathy Frizell, Hydraulic Engineer and Project Manager, D-8560, Elisabeth Cohen, Design lead, and Ernest Hall, D-8130; Donald Read, D-8420; Larry Rossi, D-8430; Dan Maag, Bill Holbert, Keith Copeland, and Craig Grush, D-8170; Mike Rasmussen, D-8160; and Steve Young, D-8311.

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Peer review was also provided by the following PN region staff:

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Dave Zimmer (PN-6520)

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Background

Total dissolved gas (TDG) levels in the Columbia River downstream from Grand Coulee Dam commonly exceed the Washington State and Colville Confederated Tribes' water quality standard of 110 percent of saturation. Exceedances are due to the combined impacts of spill operations at Grand Coulee Dam and the downstream transfer of flow with high levels of TDG generated at dams in Canada. Concerns regarding Columbia River dissolved gas supersaturation problems and potential for gas bubble disease and mortality in anadromous fish have received increasing attention over the past several years. These concerns have been raised because of the Endangered Species Act (ESA) salmon recovery efforts which utilize increased spill to accommodate fish passage through reservoir facilities on the lower Columbia and Snake Rivers and flood control spills required by above average flow conditions. The 1997 flood season required months of spill along the entire river system and the increased spill resulted in increased TDG.

The problem is flood control spills and spills requseted by the National Marine Fisheries Service (NMFS) to benefit endangered salmon migration both increase the TDG in the river because of the geometry of the hydraulic structures at Grand Coulee and other dams along the Columbia and Snake Rivers.

At the request of the NMFS, the states of Washington and Oregon waived the 110 percent TDG water quality standard in the lower Snake and Columbia Rivers during the 1995 through 1998 spill seasons. A short-term waiver was also obtained to allow voluntary spills from Grand Coulee for endangered salmon migration in the Columbia River in 1996. Temporary standards of 115 percent in reservoir forebays and 120 percent in tailwaters were adopted based on scientific evaluations which weighed the improved salmon migration conditions accomplished through increased spill against the mortality associated with gas bubble disease. The standards waiver applies only to dissolved gas conditions induced by salmon migration spills, and does not apply to flood control spills. Washington State standards contain a clause which waives the 110 percent dissolved gas standard when flows exceed the 10-year 7-day high flow, which provides some regulatory relief during flood control operations. Standards apply at the point of measurement which is located in the river 6 miles downstream from Grand Coulee Dam.

Reclamation is aware of the concerns of regional fish managers and water quality management agencies regarding potential for damage to aquatic resources downstream of the project and has been working within the NMFS regional forum to achieve long-term resolution of the problem. A number of teams have been established within the NMFS regional forum, including the Technical Management Team (TMT), the Dissolved Gas Team (DGT), the Implementation Team (IT), the System Configuration Team (SCT), and the Executive Committee (EC). These teams have been actively involved in defining and managing dissolved gas problems associated with operation of the Columbia and Lower Snake River system. The Mid and Upper Columbia River segments, including Grand Coulee Dam, Chief Joseph Dam, and the Canadian Dams are included in the system-wide TDG Management Plans. The recently formed Transboundary Gas Group (TGG) is dealing with ways to manage TDG in flows passing into and out of Canada along the Pend O'reille River and the Spokane and Columbia Rivers. Reclamation continues to be an active member of these teams through Pacific Northwest region personnel.

As a participant in the regional forum, Reclamation is working on a Gas Management Program for Grand Coulee Dam. As part of this program, Reclamation has initiated a series of outlet works operational changes, during spill, that reduce the TDG added to the river and defined a range of beneficial spillway operations. These short-term solutions are being combined with long-term efforts to determine possible structural modifications for gas abatement during spills from Grand Coulee Dam, as described in this report.

Introduction

Reclamation has been tasked in the 1998 Biological Opinion to investigate operational and structural gas abatement measures at Grand Coulee Dam. The Biological Opinion states in Chapter XII, 3.d., “The Action Agencies, in coordination with NMFS and the Regional Forum, shall jointly investigate operational and structural gas abatement measures at Grand Coulee and Chief Joseph Dams as part of system-wide evaluation of gas abatement measures. The Bureau of Reclamation shall submit an interim status report to the NMFS by April 1999 stating the findings of the investigations at Grand Coulee. The Corps of Engineers shall develop and coordinate through the Regional Forum the scope and implementation schedule for a similar investigation at Chief Joseph Dam by October 1998. The Action Agencies shall coordinate with the DGT and SCT to identify gas abating alternatives, future actions, implementation schedules, and future funding requirements for gas abatement at Grand Coulee and Chief Joseph Dams. The Action Agencies shall seek congressional authority and funding, as necessary, to implement the selected preferred alternatives.”

“Lower dissolved gas levels from Grand Coulee and Chief Joseph Dams would reduce background TDG levels caused by these projects, which may limit the duration of exposure of adult steelhead to high dissolved gas concentrations. Further, the passage survival of juvenile steelhead would be improved because increased spill would be allowed at downstream projects under the current dissolved gas cap.”

The expected filing of the final 1998 Biological Opinion led Reclamation to begin investigations into potential structural modifications to Grand Coulee for TDG abatement purposes in 1997. The study began in October 1997 and through a process of development, review, and input from the agencies in the Regional Forum, three alternatives were developed and are presented in this report. Thirty-six alternatives were originally considered and are documented in previous reports; “Structural Alternatives for TDG Abatement at Grand Coulee Dam Preliminary Concepts Report” February 1998 [1] and “Structural Alterative for TDG Abatement at Grand Coulee Dam, Conceptual Design Report,” October 1998 [2].

The three alternatives selected for feasibility study were evaluated using physical hydraulic models to determine TDG characteristics and provide design information.

Grand Coulee Dam

Grand Coulee Dam (figure 1) is located on the Columbia River about 90 miles west of Spokane, Washington. The dam was constructed from 1933 to 1942 with the forebay dam and a Third Powerplant completed in 1974. All Third Powerplant generating units were operational by 1979. The dam forms Franklin Delano Roosevelt (FDR) Lake which stretches approximately 150 miles to the Canadian border. The dam has a hydraulic height of 350 feet. The hydraulic structures are a 1,650-foot-wide gated spillway, an outlet works comprised of 40 active conduits through the dam with two tiers of 20 conduits each, original left and right powerplants on either side of the spillway, and the Third Powerplant located almost parallel to the right dam abutment (figures 2 and 3). The bottom tier of outlets is no longer in service.

The spillway is located at the center of the dam with eleven 28- by 135-foot drum gates, atop a crest at El. 1260, controlling releases up to a maximum water surface of El. 1290. The spillway design capacity is 1,000,000 ft^3/s. The spillway has a submerged roller bucket energy dissipater at El. 874.4 and discharges onto the rock surface downstream.

The 8.5-ft-diameter outlet works conduits discharge onto the downstream face of the spillway and also utilize the roller bucket dissipater (figure 4). Under normal reservoir operations, each outlet tube is capable of discharging from approximately 3,000 to 5,000 ft^3/s, depending upon the outlet elevation and the lake level. The centerline elevation of the mid-level outlets is El. 1036.67, with the centerline elevation of the upper outlets 100 feet higher. The lower-level outlets have been taken out of service and are not available for use. The design capacity of the outlet works at reservoir El. 1290 is 191,920 ft^3/s. The outlet works are generally used to lower the lake level in the spring when high runoff is expected and the lake level is below the spillway crest (El. 1260).

The powerplants have a total capacity of 280,000 ft^3/s and discharge from the reservoir to the tailrace under submerged conditions. The centerline elevation for intakes of the original left and right powerplants is at El. 1041. The left and right powerplants each contain nine 125,000 kilowatt units which in terms of discharge pass a total of about 100,000 ft^3/s. The left powerplant also houses three small station service units of 10,000 kilowatts each, for a total generating capacity of 2,280,000 kilowatts for the left and right powerplants. The Third Powerplant intake has a centerline El. 1130. The Third Powerplant has six units, three with a capacity of 705,000 kilowatts each, and three that are rated at 805,000 kilowatts each, for a total capacity at Grand Coulee of 6,809,000 kilowatts. The Third Powerplant is capable of passing 180,000 ft^3/s when generating power. When not generating power, the left and right powerplant turbine units can pass 500 ft^3/s each and the Third Powerplant turbines 3,000 ft^3/s each, for a total speed-no-load capacity of 27,000 ft^3/s. The tailwater depth varies for a normal powerplant discharge range from about 80 to 100 feet referenced to the invert of the roller bucket.

The Grand Coulee Pump-Generating Plant consists of six pumping units and six pump generators, which lift water to irrigation facilities to the south of the river. The pump generators may also be used to generate power during peak power demand periods, at a capacity of 50,000 kilowatts each. The pump-generating plant intake is located at centerline El. 1193.27. The extensive irrigation works of the project extend southward on the Columbia Plateau, 125 miles to the vicinity of Pasco, Washington, where the Snake and Columbia Rivers join.

The geometry of the hydraulic structures at Grand Coulee has a major influence on the gas transfer characteristics at the dam and makes addressing the TDG issue more complicated than at many of the lower dams on the Columbia and Snake River systems.

TDG Evaluation for Existing Conditions

The analysis used in the TDG evaluation is discussed in this section. The mixing of flows with differing TDG levels and the existing operational data used in this analysis was reported fully in the previous document "Operational Alternatives for Total Dissolved Gas Management at Grand Coulee Dam" [3]. The operation of the existing spillway is documented in that report [3] and is not discussed in this report.

The increase in TDG below hydraulic structures is due to entrainment of air along the spillway face or outlet jets in atmosphere and at the location where the flow enters the plunge pool or stilling basin. This air is entrained as bubble, which have a high surface area for air-to-water mass transfer. When these bubbles are pulled to depth, the bubble-water equilibrium increases proportionate to the pressure. Thus, relative to atmospheric conditions, supersaturation of dissolved gas is possible.

Flow Mixing

The location of all the hydraulic structures at the dam with respect to each other, the tailwater pool, and the river channel influence tailrace mixing and consequently local TDG concentrations. The spillway and outlet works releases travel down the face of the dam and plunge into the roller bucket energy dissipater at the base of the spillway. The tailwater depth in conjunction with the spillway or outlet works releases provide the opportunity for a deep plunge depth during normal operation that increases potential for supersaturation.

The spillway and spacing of the outlet works conduits across the spillway face are wider than the river channel. The outlet works conduits on the right or east side of the spillway are used most often because they are the best aligned with the river channel (figure 3). The left powerplant discharges to the left or west of the spillway and has a capacity of about 50,000 ft^3/s. This flow is relatively isolated from the main tailwater pool, particularly when the spillway is operating. The right powerplant, also with a capacity of about 50,000 ft^3/s, discharges adjacent to the spillway but normal to the discharge from the Third Powerplant. The Third Powerplant has a capacity of about 180,000 ft^3/s and discharges almost parallel to the original dam axis and normal to all the other hydraulic structures and the river channel. The large capacity of the Third Powerplant highly influences the flow conditions in the tailwater pool and in the river channel downstream. Third Powerplant use is preferred by Bonneville Power Administration (BPA) and the dam operators because of operational issues. Flow generally crosses the tailwater pool and travels along the left river bank for quite a distance downstream. During an investigative trip [3], it was determined that combined release flows were not fully mixed at a distance of 1½ miles downstream with a total flow of about 110,000 ft^3/s. With higher flows, the distance for complete mixing is expected to be even longer.

The Third Powerplant flow, which is not aerated, discharges far enough away from the roller bucket that its releases should not be entrained and supersaturated by outlet or spillway releases. The adjacent left and right powerplant releases have a greater probability of mixing with and being supersaturated by the outlet or spillway releases. This was not evaluated in the feasibility stage but should be in a final design through the use of a large scale 3-dimensional physical model and potentially near field studies as they may apply.

The combined flow, both spill and powerplant releases, travels down river about six miles to the TDG fixed monitor, GCGW, located out in the river about 20 feet from the left bank and at a depth of about 15 feet. The flow is fully mixed by this point [3] and some degassing has occurred with travel to this location. The TDG percent at the fixed monitor was developed in reference [3] and is described by the following equation:

$$\%TDG_{GCGW} = \frac{Q_{OW}(\%TDG_{OW}) + Q_{SPWY}(\%TDG_{SPWY}) + Q_{POWER}(\%TDG_{POWER})}{Q_{OW} + Q_{SPWY} + Q_{POWER}} \qquad (1)$$

This equation combines the potentially different TDG concentrations produced by outlet works or spillway releases, power generation, and the corresponding discharge volumes. Degassing in the river channel is not well defined. This analysis doesn't include degassing between the dam and the fixed monitor and is therefore conservative.

Existing Outlet Works TDG Generation

Operation of the existing outlet works increases the TDG levels below Grand Coulee Dam. The level of the increase has been documented in the report "Operational Alternatives for Total Dissolved Gas Management at Grand Coulee Dam," [3]. The 40 outlets are located on the face of the dam in 20 pairs at two elevations. Each pair is separated by 50 ft concrete block sections starting adjacent to the right powerplant. The outlets are used to make flood releases when the reservoir is below the level of the spillway crest at El. 1260. The spillway releases produce less TDG than the outlet works releases because spillway release flow depths are small and much less concentrated than the outlet works releases. Spillway releases should be used whenever possible. The focus of this report is on the existing outlet works TDG generation and how their productions may be improved with the proposed modifications.

The TDG levels generated for the total combined releases, associated with outlet works releases for the design spill of 50,000 ft^3/s, varies with total release magnitude and with TDG levels in the powerplant releases (which are the same as the reservoir TDG levels). In the concept report, the 50,000 ft^3/s design discharge was combined with powerplant releases of 160,000 ft^3/s and lake TDG levels of 100 and 120 percent. The following tables expand that analysis to provide direct comparison to the structural alternatives under consideration for the outlet works under a wider range of discharges and lake TDG levels. It was assumed that the TDG level of the outlet works releases was 190 percent and was mixed with powerplant releases using equation 1. TDG levels were computed for the exact 50,000 ft^3/s design discharge using the mid-level outlet works only with reservoir water surface Els. 1208 and 1260. The mid-level outlet works produce somewhat different discharge values with the various heads. The graphical representations of the resulting combined TDG in the river are shown as figures 5, 6, and 7. Currently, outlet works releases will never reach the 110 percent standard even when mixed with powerplant flows and lake TDG levels of 100 percent.

Table 1. - TDG levels for the existing outlet works releases at reservoir elevation 1208 and 1260 with various lake TDG levels.

	Qs	Qpp	Qt	TDG of OW spill	Combined TDG in the river downstream from Grand Coulee Dam						
Design Q = 50,000 cfs	cfs	cfs	cfs	%	100%	105%	110%	115%	120%	125%	130%
Min. tw El. 958	50000	25000	75000	190.00	160.00	161.67	163.33	165.00	166.67	168.33	170.00
Normal tw range El. 962-966	50000	72000	122000	190.00	136.89	139.84	142.79	145.74	148.69	151.64	154.59
Max. tw range El. 966-970	50000	118000	168000	190.00	126.79	130.30	133.81	137.32	140.83	144.35	147.86
Tw El. 970-972	50000	160000	210000	190.00	121.43	125.24	129.05	132.86	136.67	140.48	144.29
7Q10 tw El. 975	50000	191000	241000	190.00	118.67	122.63	126.60	130.56	134.52	138.49	142.45
Tw El. 980	50000	245000	295000	190.00	115.25	119.41	123.56	127.71	131.86	136.02	140.17
Tw El. 985	50000	280000	330000	190.00	113.64	117.88	122.12	126.36	130.61	134.85	139.09
6 blocks operating for mid operation @ reservoir El. 1208											
6 blocks Qs El. 1208	53800										
Min. tw El. 958	53800	21200	75000	190.00	164.56	165.97	167.39	168.80	170.21	171.63	173.04
Normal tw range El. 962-966	53800	68200	122000	190.00	139.69	142.48	145.28	148.07	150.87	153.66	156.46
Max. tw range El. 966-970	53800	114200	168000	190.00	128.82	132.22	135.62	139.02	142.42	145.82	149.21
Tw El. 970-972	53800	156200	210000	190.00	123.06	126.78	130.50	134.21	137.93	141.65	145.37
7Q10 tw El. 975	53800	187200	241000	190.00	120.09	123.98	127.86	131.74	135.63	139.51	143.39
Tw El. 980	53800	241200	295000	190.00	116.41	120.50	124.59	128.68	132.77	136.85	140.94
Tw El. 985	53800	276200	330000	190.00	114.67	118.86	123.04	127.23	131.41	135.60	139.78
5 blocks for mid operation @ reservoir El. 1260											
5 blocks Qs El. 1260	50300										
Min. tw El. 958	50300	24700	75000	190.00	160.36	162.01	163.65	165.30	166.95	168.59	170.24
Normal tw range El. 962-966	50300	71700	122000	190.00	137.11	140.05	142.98	145.92	148.86	151.80	154.74
Max. tw range El. 966-970	50300	117700	168000	190.00	126.95	130.45	133.95	137.46	140.96	144.46	147.96
Tw El. 970-972	50300	159700	210000	190.00	121.56	125.36	129.16	132.96	136.77	140.57	144.37
7Q10 tw El. 975	50300	190700	241000	190.00	118.78	122.74	126.70	130.65	134.61	138.57	142.52
Tw El. 980	50300	244700	295000	190.00	115.35	119.49	123.64	127.79	131.94	136.08	140.23
Tw El. 985	50300	279700	330000	190.00	113.72	117.96	122.19	126.43	130.67	134.91	139.15

Qs refers to the spilled volume through the outlet works, Qpp refers to the flow through the 3rd powerplant, and Qt is the total flow in the river. OW refers to the outlet works. 7Q10 refers to the 7-day 10-year event.

Feasibility Design Discharge and Tailwater

A design discharge for TDG evaluation was developed during the preliminary concept phase in February 1998 [1] and is still being used for the feasibility phase. The design discharge for the preliminary concept phase was based upon the 7-day, 10-year event (7Q10) minus an average base powerplant flow using hydrologic data from 1975 to 1997. The 7Q10 event is the highest flow rate that occurs for 7 consecutive days, once every 10 years. The 7Q10 event for the concept phase was determined to be 210,000 ft^3/s. An average base powerplant flow was determined of 160,000 ft^3/s. The 50,000 ft^3/s design flow was estimated as the difference between the base 7Q10 event of 210,000 ft^3/s and a base powerplant flow of 160,000 ft^3/s. In June 1998 the alternatives under investigation were narrowed from nine in the preliminary concept phase to five for the concept phase. The method used to determine the 7-day 10-year design flow value for the Upper Columbia River was still under discussion and 50,000 ft^3/s continued to be used as the design flow value for Grand Coulee conceptual design studies. In October 1998, the five conceptual alternatives were presented to the SCT for their review and comment regarding reducing the number of alternatives under consideration to three for proceeding into the feasibility studies. The design discharge value for Grand Coulee Dam during the feasibility studies was also discussed. No formal method to compute the design discharge was determined but a consensus was reached to continue into the feasibility phase with a design flow rate of 50,000 ft^3/s. The feasibility designs proceeded using 50,000 ft^3/s as the design discharge for sizing the three selected alternatives.

The method used to determine the 7-day, 10-year design discharge has since been defined and the value of 241,000 ft^3/s determined for both Grand Coulee and Chief Joseph Dams [4]. At the time of this report, the “credit” or the amount of the design discharge that will be passed by the powerplants at Grand Coulee was still under discussion, therefore, 50,000 ft^3/s continued to be used as the design flow throughout the studies. Flexibility of each alternative to be enlarged or reduced in size to accommodate a different design value as needed is provided.

The tailwater elevation is critical to the TDG evaluation for each of the alternatives. Because Grand Coulee Dam operates to supply peaking power demands the potential range of tailwater operation is large. There is an operating restriction that limits tailwater fluctuation during any 24-hour period to 22 feet to maintain slope stability in the riprap channel below the dam. In addition, the reservoir behind Chief Joseph Dam downstream from Grand Coulee is normally operated between Els 950 to 956 during the spill season which influences Grand Coulee, tailwater elevations. Tailwater data from the river gage located at the bridge ½-mile downstream from the dam, was obtained from the Corps of Engineers (COE) web site for 1997. Large flows in the river in 1997 provided a large range of data for the tailwater curve, figure 8. There is substantial scatter in the data. This scatter is most likely caused by reservoir fluctuations at Chief Joseph Dam and the location of the gage which may be influenced by the high-flow velocity exiting the pool below the dam and near the gage site.

With the tailwater curve, the elevations associated with various spill and powerplant releases can be determined, however, the spill may vary over an extremely large range of powerplant releases. The 7Q10 event of 241,000 ft^3/s yields a tailwater elevation of about 975 feet. However, the base flow from the powerplant releases may vary from 30,000 ft^3/s to 280,000 ft^3/s (> the 7Q10) and spill may be required at any time. With the design flow rate of 50,000 ft^3/s, the corresponding tailwater could vary from a minimum of El. 960 to a maximum of El. 985 for total flow rates in the river of 80,000 to 330,000 ft^3/s. This is a possible tailwater variation of 25 feet.

Recent spill records for the combined spillway and outlet works releases show that the flow rate varied from about 100,000 to 260,000 ft^3/s during the 1996 and 1997 spill seasons [3]. This indicates that the design tailwater for the outlet works could vary from about El. 958 to El. 978 using the maximum scatter in the tailwater plot over a range of 100,000 to 260,000 ft^3/s. This is a possible tailwater variation of 20 feet.

The minimum tailwater used in this study was set at El. 956, based on the recognition that spill will generally only occur when flows are high in the system and will likely be the reservoir elevation of Chief Joseph Dam.

To assist with further defining the tailwater elevation range, the average daily flow values for 1995 to Dec 1998 were obtained from the COE web site and investigated with a histogram, figure 9. The data was slightly skewed from normal, but was used to determine the "normal" flow rate expected in the river. The average flow rate was 121,790 ft^3/s with a standard deviation of ±46,340 ft^3/s Using the tailwater information from figure 8, the flow range of 75,450 to 168,130 ft^3/s also produced a range of tailwater elevations. The evaluated tailwater range using figure 8 is El. 958 to El. 961 for 75,450 ft^3/s, El. 962 to El. 966 for the average flow rate of 121,790 ft^3/s, and El. 966 to 970 for 168,130 ft^3/s. Therefore, the maximum range of tailwater expected during normal operation would be El. 958 to 970 feet or 12 feet. Feasibility designs were developed by focusing on this tailwater elevation range in addition to the 7Q10 tailwater of El. 975. Operation outside the expected range of tailwater elevations may reduce TDG abatement effectiveness.

Feasibility Designs for Structural Alternatives

The following sections describe the development of the three alternatives that were selected for physical hydraulic modeling, feasibility designs, and cost estimates. The three alternatives investigated through the feasibility level are:

- Cover and Extend Mid-level Outlets for Submerged Releases (Alternative 1)
    - Objective to transfer TDG levels from the lake to the river by preventing surface turbulence in the release which would re-entrain air.
- Deflectors - Minimal Number of Outlets (Alternative 3)
    - Objective to reduce TDG production by minimizing the plunge depth from the outlets, thus lowering TDG below those produced by the existing condition.
- Forebay Pipe with Cascade (Alternative 5)
    - Objective to decrease TDG levels from those in the lake by reducing the TDG in the spill volume by creating small flow depths, stripping, and preventing replunging to depth.

Each section contains a description of the respective alternative, updated drawings showing components and dimensions, additional information regarding the hydraulic behavior, expected TDG performance evaluation, and refined cost estimates. The alternatives were previously numbered 1, 3 and 5 from the five alternatives that were outlined in the conceptual design report [2] and this numbering is maintained in this report for ease of reference with the earlier reports [1].

The description of each alternative has been updated from the conceptual stage to reflect the further level of study associated with Reclamation feasibility designs. The initial design components were modified based on information produced by the hydraulic model study and additional construction considerations. The feasibility designs were based upon the results of the hydraulic modeling efforts to optimize the reduction of the TDG production of each alternative. For alternatives 1 and 3, the cost estimates were prepared per block modified to obtain a cost for various numbers of blocks to pass various discharges. This allows flexibility in determining the design discharge for decision-making purposes. This also provides an indication of the complexity and variability of the alternatives.

Hydraulic Modeling

The hydraulic modeling for the feasibility level structural alternatives was conducted in 1999 and 2000. The modeling was used to evaluate the potential for each of the alternatives to perform as predicted in the concept phase [2]. The challenge was to apply model scales adequate for investigating the hydraulic design while having a realistic size for constructing the models. Ideally, a 3-dimensional model of the dam, spillway, outlets, powerplants, and downstream pool and river channel would be the best way to investigate the total effect of each of the alternatives over the entire range of flow conditions. The large flow rates, high heads, and spatial extent of the separate alternatives created problems when trying to combine the investigation of several alternatives into one model.

Each of the three alternatives produces unique problems when trying to analyze the hydraulic features and flow conditions that need to be modeled. A three dimensional model that included all components of the dam and the proposed alternatives would produce very small hydraulic structures with a large model scale. Such a model would not allow effective evaluation and development of the alternatives. It was apparent that separate models would be needed.

Small model scales are important in determining aspects of TDG performance as well as other reducing other scale effects such as surface tension. The TDG performance was evaluated qualitatively in the models primarily by visual observations of jet mixing, shear zones, average flow depths, jet plunge depths, and, where possible, actual entrainment of air bubbles. Visual observations were aided by the use of clear side walls in the models and use of fiber tufts and dye to define mixing depths and velocity directions. The TDG levels were then assigned based upon the assumption that air bubbles would be entrained to the depths observed in the models. Because turbulence intensities in the model are less than in the prototype structures, true evaluation of surface gassing could not be evaluated for certain. Therefore, the TDG values from the models are most likely conservative.

Two models were constructed to study:

- The existing outlet works flow conditions and the alternatives involving the outlet works; the mid-level cover and extension and the deflectors (Alternatives 1 and 3).
- The forebay pipe with cascade (Alternative 5).

Outlet Works Model

The sectional outlet works model represented one pair of the ten pairs of upper- and mid-level outlets at the dam. The model was initially constructed with the existing outlet and flip bucket geometries and included a tailbox to allow investigation of a wide range of tailwater elevations.

The outlet works model was constructed using Froude similitude to a 1:17 scale to effectively represent gravitational influences. The scale was chosen to enhance the size of the model as much as possible, once the sectional model was decided upon. The height and length of the model were the restricting factors. The largest scale possible was chosen to fit the model into laboratory space and to represent turbulence intensity and bubble dynamics as accurately as possible. TDG transfer itself was not measured in the model. The scale is sufficiently large to allow use of entrained bubble observation to define the vertical extent of the effective gas transfer zone. Froude law similitude produces the following relationships for the 1:17 model:

$$\text{Length ratio} = L_r = 17{:}1$$

$$\text{Velocity ratio} = (V_r)^{½} = 17^{1/2} = 4.123{:}1$$

$$\text{Discharge ratio} = (Q_r)^{5/2} = 1{,}191.58{:}1$$

The reservoir was modeled using 2 pressure tanks, one for each outlet works intake. The 8.5-ft diameter outlet conduits were modeled with 6-in ID pipe, one in plastic to allow viewing of the flow conditions. The prototype gates were modeled, but were only operated in the fully open or closed positions. The conduits daylighted onto the 0.8:1(H:V) sloping face of the dam with a 10° flatter slope. The flow entered the tailwater and plunged to or near the invert of the 50-ft-radius flip bucket energy dissipater that was modeled in concrete with a 35.3 in radius. The model included a straight section that extended 284 ft downstream from the lip of the flip bucket.

Forebay Pipe with Cascade Model

Froude similitude was also used for scaling the 3-dimensional forebay pipe with cascade model. The large width of the cascade with the relatively small drop and the small size of the control gates made scaling difficult. A model scale, of 1:43.6364, was chosen to fit within an existing box and to produce a standard size exit pipe for the manifold section. The scale provided adequate pipe sizes, model width and extent to investigate flow depths and replunging characteristics, however, turbulence levels would be difficult to determine, thus producing conservative results.

Froude law similitude produced the following relationships:

$$\text{Length ratio} = L_r = 43.6364{:}1$$

$$\text{Velocity ratio} = (V_{r)}^{½} = 43.6364^{1/2} = 6.6058{:}1$$

$$\text{Discharge ratio} = (Q_{r)}^{5/2} = 43.6364^{5/2} = 12{,}578.3{:}1$$

The model included a head tank to simulate the reservoir, a section of the 40-ft diameter pipe supplying the 800-foot-long manifold gate chamber, the exit pipes, gates, stilling well, stepped cascade and tailwater pool. The 800-foot-long manifold was modeled by an 18.33-ft long rectangular sheet metal box. The exit 12-ft to 7.5-ft pipes were modeled with 3.3-in to 2.0625-in I.D. pipes. The 50-ft wide by 102-ft deep stilling well was modeled by a 1.145- by

2.34-ft deep plywood section leading to the originally stepped cascade. The stepped cascade was modeled with a flat crest leading to steps on a 2:1 (H:V) slope that changed to a 6:1 sloping section to carry the flow underneath the tailwater.

Both models were calibrated using the permanent laboratory Venturi systems to measure the inflows. Pressures were measured downstream of the head tanks to measure head for each flow rate tested. Point gages in the tailboxes recorded tailwater elevations.

The results of the hydraulic model studies were used in the structural TDG feasibility designs of the alternatives.

Cover and Extend Mid-level Outlet Works (Alternative 1)

Description

The intent of this alternative during the concept phase was to transfer the TDG levels occurring in the reservoir to the river downstream without increasing TDG levels, similar to that which occurs with powerplant releases. Potential increase in TDG would be eliminated by modifying the mid-level outlets for submerged releases and changing control to the downstream end of the pipe thus preventing air from entering the outlets after initial start up, figure 10. Using the conceptual level designs a pair of outlets in one block with approximately ½ of the neighboring solid blocks on either side was constructed and tested in the 1:17 scale outlet works model.

The existing outlet works is currently controlled either at the 102-inch ring seal gates or at the outlet works release point where the 8'-6" pipe reduces to 7'-9". The gates are either open or closed and are not used for control at partial gate openings. The existing mid-level outlet conduits are steel lined. An air vent supplies air to the conduit downstream of the ring seal gates. The outlet works discharge highly aerated flow onto the face of the dam which plunges to the depth of the tailwater, figure 2, producing high TDG saturation levels.

Modifying the outlets for submerged release prevents air entrainment and the associated high TDG levels as the flow plunges to depth. To ensure that the extended pipe did not entrain air and was fully pressurized, the exit of the conduit is submerged and the cross-sectional area reduced to maintain flow control over the expected range of reservoir operation. The control at the downstream end of the pipe is achieved through the use of an in-line steel diffuser section. The diffuser also dissipates additional energy. The existing 1-ft-diameter air vents downstream from the ring-seal gates will be modified and the area used to facilitate the transition from open channel flow to pressurized flow in the conduit. The air vents will be replaced with 1-ft-diameter air relief valves installed in the gate chamber on the downstream side of the gates to allow air to escape during pipe filling. There will be some excavation of concrete required in the gate chambers to facilitate this modification.

Encapsulation and extension of the outlets is necessary to prevent air entrainment and avoid affecting spillway flows. To accomplish the extension, the conduits were encased behind the concrete face of the spillway. The conduits were turned an additional 10E producing a 51E20' angle over a 39'-8" radius to align the conduits behind the 0.8:1 sloping dam face (figure 10). Addition of the extension of outlet works pipeline, including encasement of the outlet works pipe, the diffuser, and the bend will require concrete excavation. Extending the 8'-6" diameter outlets, 128'-5" long down the dam face, providing room for the construction of the diffuser, and releasing into the spillway roller bucket, requires excavation of trenches 16'-11" deep by

33'-6" wide on the concrete face of the dam, figure 10. The pipe extension will be covered by a minimum 5-foot thickness of concrete and have an 8'-6" diameter steel lining for the entire extension, including the diffuser. There will be two layers of reinforcement around each conduit that extend down the face of the dam. Anchors will connect the modification to the existing structure. The 18'-4" long diffuser was designed and tested after initial studies indicated that the 6'-0" diameter reducer would not be acceptable. The 6'-0" diameter reducer produced a high-velocity jet resulting in large recirculation areas and surface turbulence that entrained additional air to the depth of the roller bucket and caused higher TDG levels.

The diffuser open area was selected to produce downstream control, accommodate reservoir evacuation criteria, and prevent debris clogging. The open area in the diffuser was sized to pass the 50,000-ft^3/s design discharge under the minimum operating reservoir El. 1208 while maintaining downstream control. The diffuser includes 220 six-inch diameter holes in 11 rows placed around the circumference of the 8'-6" diameter stainless steel diffuser. The diffuser holes were sized to pass debris that might pass the trashracks on the pipe intake. The diffuser discharges into a 14-ft deep by 29-ft long chamber excavated into the dam and flip bucket. The chamber provides for 2'-9" of clearance around each diffuser. The chamber will be stainless-steel lined to reduce maintenance and prevent abrasive erosion of the concrete. The diffuser will be stabilized and supported with a steel beam across the bottom. The diffuser could be removed if necessary but the use of special equipment and divers would be required. The downstream half of the top of the chamber is ramped up on a 10 degree angle away from the face of the dam to allow flow from the diffuser to expand exiting from the chamber and to divert spillway flows away from the chamber area on the surface. The initial section of the flip bucket includes a 29-foot-wide, 70 foot radius from the chamber to a 14-ft, 9-in long, flat section to the invert of the existing bucket. Vertical walls extend between the flatter flip bucket radius in the modified section and the existing flip bucket radius that is tangent to the face of the dam. The lower, inoperable outlet works conduits will be backfilled for a short distance to ensure they remain plugged and to support the conduit extensions.

The modification would be made in pairs of outlets because the outlets are built in pairs and construction considerations dictate the paired approach. A total of eight pairs of outlet works would be modified to pass the minimum feasibility design flow of 50,000 ft^3/s at reservoir water surface El. 1208. This alternative can be modified to accommodate a larger or smaller design discharge easily. The limitations are a minimum of one pair of outlets producing a flow rate of 6,276 ft^3/s to all ten pairs of outlets with a discharge of 62,700 ft^3/s.

Maintenance Issues

Before modeling the diffuser, the issues of debris and maintenance were discussed. The debris issue was addressed by sizing the diffuser holes to be as large as the trashrack openings. It was reasoned that the diffuser should be able to pass any debris that would pass the trashrack structures upstream. The diffuser will be made of stainless steel to prevent rusting, abrasion damage, and reduce maintenance requirements. In addition, the chamber opening and location of the diffuser allow it to be removed and replaced if necessary. The inspection and maintenance of the diffuser will require the use of divers.

Hydraulic and Total Dissolved Gas Evaluation

Hydraulic Evaluation

The hydraulic model was used to determine the affect of changing the outlet works discharge characteristics from free flow plunging into the tailwater to pressurized submerged flow into a modified flip bucket. The model was used to determine discharge capacity of the modified outlets, pressures in the outlet pipes, diffuser and dissipation chamber performance, flip bucket modifications, and downstream flow conditions.

To maintain pressurized flow in the extended conduit, the downstream exit area was reduced at the diffuser. The existing capacities of the mid-level outlets are 5336 ft^3/s at El. 1290 and 4486 ft^3/s at El. 1208 per outlet (not per pair). The outlets currently discharge freely onto the face of the dam without influence from the tailwater. The appropriate reduction in pipe diameter or exit area considered the required operation under current flood control operation and reservoir evacuation for emergencies. The smaller area required to maintain control at the pipe exit reduces the maximum discharge capacity from the outlet works.

The diffuser open area and the head differential between the reservoir and the tailwater submergence control the discharge through the outlet works. The discharge is therefore a function of the differential head, diffuser open area, and diffuser orifice discharge coefficient. Each diffuser opening was shaped like a sharp edged orifice. Figure 11 shows the relationship developed by the model tests between the differential head and the discharge per outlet over the range of reservoir and tailwater elevations tested in the model from El. 1290 to El. 1208 and El. 960 to El. 985, respectively.

The discharge per outlet may be determined by the following equation:

$$Q_{outlet} = (\Delta H + 242.72) / 0.1484 \qquad (2)$$

where ÄH = difference between the reservoir and tailwater elevations (ft). This equation was then used to replot the discharge relationships as a function of the tailwater. With a known reservoir elevation and tailwater elevation, the discharge per outlet can also be determined from figure 12. For example, the discharge per outlet for reservoir El. 1260 and tailwater El. 960 (Ä H=300) is 3657 ft^3/s. Equation 2 or figure 12 may be used to open or close the appropriate number of outlets to attain the desired releases.

The normal operation at Grand Coulee Dam is to set the outlet works releases, then change the powerplant releases to match demand. This would lead to changing tailwater elevations under basically constant reservoir head. The change in discharge produced by a 5 foot change in the tailwater elevation, for a given reservoir elevation, is 33.7 ft^3/s per outlet, shown by figure 12.

Using equation 2, at minimum reservoir El. 1208 and maximum expected tailwater El. 985, which corresponds to a powerplant flow of 280,000 ft^3/s plus the design releases of 50,000 ft^3/s, the discharge per outlet is 3138 ft^3/s. To pass the design flow rate of 50,000 ft^3/s, 16 outlets or 8 blocks will require modification.

The diffusers discharge into the dissipation chamber, refer to figure 10. By continuity, the average velocity exiting the chamber from a pair of outlets will range from 15 to 19 ft/s. The flow exits the chamber both along the invert and vertically through the water column above the flip bucket invert. The majority of the flow, however, exits along the invert. Flow exiting the chamber along the invert remains within the 29-ft-wide modified bucket section until reaching the invert of the bucket. The flow then spreads laterally across the existing flip bucket. The lateral spread from the invert to the end of the existing flip bucket is about 50 ft and will combine with flow from adjacent outlets. Flow exiting the dissipation chamber into the water column above the bucket has both downstream horizontal and vertical components. This flow helps disperse the upward velocity component from the flip bucket. Still, the combined flow condition produces a dominant downstream and upward velocity field, forming a turbulent boil at the surface just downstream from the bucket.

Figure 13 is a view of the surface of the tailwater in the model. In addition to providing control, the diffuser increases exit jet diffusion, thus decreasing downstream surface turbulence and producing a water surface appearance similar to powerplant releases. The diffusers are submerged, but submergence depends on the tailwater elevation at the time of the releases and the relationship between the elevations of the outlet exit, the roller bucket invert and/or the invert of the river bed downstream from the roller bucket. Minimum submergence with respect to the invert of the roller bucket will be 82 feet for tailwater El. 956. Submergence under the expected tailwater of the total design discharge is 98 feet. Referenced to the river bed El. 900, the minimum and maximum submergences would be 56 and 72 feet, respectively.

Typical flow conditions in the model are shown in figures 13, 14, and 15. The diffusers were submerged, however, flow velocities and orientation produced some surface turbulence and vertical mixing (figures 14 and 15). Vertical mixing tends to produce air entrainment to shallow depths. The flow recirculated back towards the dam along the sides of the tailbox and in front of the outlets. Side recirculation should be minimized when adjacent outlets are operating. Surface recirculation was not strong enough to form air entraining surface vortices.

The hydraulic performance of the flip bucket was confirmed by the model study, however, additional computations were made to check reliability. The minimum discharge for the paired modified outlets is approximately 7,853 ft^3/s. Assuming the diffusers spread the flow evenly across the 29 foot wide opening in the block, this computes to a 270 ft^3/s per linear foot unit discharge. Assuming that the roller bucket was designed for the maximum spillway discharge, these unit discharges would be compared to a maximum spillway unit discharge of 606 ft^3/s per linear foot. Therefore, the roller bucket should continue to produce acceptable hydraulic performance, with the flip bucket modification. In addition, turbulence intensity should be reduced compared to the existing outlet works discharge because of the smaller unit discharge with the modified outlet capacity. Flow exiting the flip bucket should impact the riprap channel downstream less than the existing outlet works operation.

Reservoir Evacuation

Evacuation concerns were addressed in this design and will be evaluated further in the final design. The evacuation criteria for FDR Lake can not be met unless the powerplant outlets are used in addition to the outlet works for evacuation. The capacity of the outlet works is relatively small compared to the powerplant outlets and the reduction in capacity due to the diffuser does not make a significant difference in the evacuation of the reservoir. The capacity of the mid- level outlets was only reduced by approximately 28 percent. The capacities of the upper- level outlets, the spillway, and the powerplants were not reduced. The reservoir can be drawn down within Reclamation guidelines using the Third, Left, and Right Powerplants, the outlet works, and spillway for evacuation. An evacuation study was performed assuming a maximum total outflow of 500,000 ft^3/s. Using all the powerplant outlets but with no more than 195,000 ft^3/s average per day (which is approximately 70 percent of capacity), a high average inflow of 200,000 ft^3/s, and the outlet works, the reservoir could be drawn to El. 1180 in 18 days. Lower inflows would result in a smaller dependance on use of the powerplant outlets.

Pressure Results

The model was instrumented with piezometer rings and taps on the invert and crown to measure hydrostatic pressure heads throughout the system. The upstream ring was used to determine the upstream hydraulic grade line. A spreadsheet computation was made back to the reservoir that included losses for the existing outlet works geometry to calibrate the model under known flow conditions. With the model calibrated, the rating could be developed for the restricted area of the diffuser as discussed previously. In addition, the piezometers showed no low pressures throughout the pipeline. Both the spreadsheet computations and the piezometer data indicated that the pipes would be positively pressurized once the air was evacuated. The pressures measured in the pipeline model at the elbow were about 150 to 170 feet. There was no evidence of low pressures that would cause cavitation damage. A large pressure drop occurs across the orifices at the diffuser, breaking head, and thus reducing tailrace turbulence.

Of special note, the pressures will vary greatly during filling of the pipe. In the model, the tailwater was below the exit of the pipe during initial startup. This produced large pressure fluctuations in the pipe and will produce additional saturation of the tailwater until the air has evacuated from the pipe. In the prototype the pipe exit will be submerged during initial opening of the gates and the same phenomenon will occur. However, the magnitude should be less, because the pipe will be partially full.

In the concept design, a water surface profile and hydraulic grade line program, CTAC, was used to model the pressure flow for the extension of the outlet works modification and the water surfaces for the deflector alternatives. For feasibility design, the hydraulic grade line was computed for the outlet works extension using a spreadsheet with assumed losses. This computation indicated the pressure drops in the pipe as the pipe follows the downstream face of the dam, but a positive head is present at all points along the profile. Model results confirmed that the pressure remained positive throughout the conduit profile with backpressure maintained by the diffuser.

The velocity of each jet issuing from each diffuser port was computed. The velocity ranged from a maximum of 90 ft/s under reservoir El 1290 and tailwater El 956, to a low of 72.5 ft/s with Res. El 1208 and tailwater El 985. Using equation 3 [5]:

$$V_m = 6.2 V_0 D / x \qquad (3)$$

the distance, x, required for the diffuser released jets to begin to diffuse is about 5 ft under maximum head and tailwater differential. Therefore, there will be a high impact on the sides of the chamber that are located 2.75 feet from the edge of the diffuser. A steel liner was designed for the chamber to protect it from the expected high impact velocities.

Total Dissolved Gas Evaluation

The objective of this alternative was to transfer the TDG levels from the reservoir to the tailwater pool. This would require preventing air entrainment as flow passes through the pipe from the reservoir to the tailwater pool. This requires that the air must be removed from the conduits during initial opening or filling of the conduits, the submergence be adequate, and there are no strong circulation patterns in the roller bucket and tailwater.

The outlet gates will be operated as they currently are - either fully open or fully closed. Air will uptake during the opening and filling, and during the closing and emptying process. This cannot be avoided since control switches from the gate to the downstream diffuser, and the flow changes from free surface flow to fully pressurized pipe flow. This process will cause a period of rough operation that will exist as the flow transitions between free and pressure flow and air is released through the air relief valves. Air will be released downstream when the pipe is filling and additional supersaturation will occur for a short duration. After pressurized flow is attained, no further air entrainment should occur in the conduit.

The exit reducer proposed in the concept phase was found to concentrate the jets released from the paired outlets and produced a severe recirculation zone in the flip bucket. The jet exited the flip bucket with a high vertical component that generated a large surface boil with substantial surface air entrainment. The entrained air recirculated upstream and was drawn to the depth of the flip bucket by the flow along the shear zone. It was apparent that to reduce or eliminate air entrainment, the energy in the jet and the turbulence intensity of the jet needed to be reduced. The resolution of the problem was the addition of the in-line diffuser. The diffuser produced a large reduction of energy that greatly reduced the velocity of the vertical component of the jet exiting the bucket. The dissipation chamber also redistributed the flow from the diffusers and spread the flow into the bucket more widely and uniformly. This also reduced the vertical velocity component leaving the bucket. Use of the diffuser, however, did not fully eliminate the problem of surface turbulence that would lead to surface air entrainment and increased saturation.

Flow visualization techniques were used to determine the flow patterns and evaluation of the potential TDG levels that would be produced by this alternative, figures 13, 14, and 15. Dye injection, underwater lighting, fiber tufts, and movement of loose material in the bucket were all used to evaluate the flow conditions and TDG potential. Figures 14 and 15 show the strength, direction, and fluctuations of the velocity component as demonstrated by the movement of the fiber tufts in the photographs. The diffuser is off the pictures to the right side and flow is from right to left. Observations of these flow conditions indicated that surface recirculation occurred, but did not extend to depth. Vertical mixing was indicated by the dye tracings for shallow depths that would entrain air in the prototype.

In addition to flow observations, TDG potential was also investigated by comparing the diffuser flow conditions to the existing powerplant tailrace flow conditions [6]. This included comparison of the flow magnitudes, recirculation, submergence depth, and exit velocities. Typical draft tube exit velocity from the Third Powerplant would be around 13 ft/s. The jet would be angled on a 6:1 slope into the tailwater pool. Exit velocity from the diffuser chamber would be about 15 to 19 ft/s on a 1:1 slope from the existing bucket invert. The surface turbulence generated by flow from the diffuser will be of slightly higher magnitude and will be oriented more vertically than typical powerplant releases. Therefore, some surface entrainment is expected. These observations indicated that the design will not fully prevent air entrainment and gas transfer levels were predicted equivalent to the depth of the vertical mixing observed in the model.

Extending and covering the outlet works for submerged discharge will generate enough surface turbulence to entrain air to various depths depending upon the available energy head. Greater surface turbulence was observed in the model with higher reservoir heads and lower tailwater elevations. The TDG levels associated with the design spill of 50,000 ft^3/s at reservoir El 1208 would require eight blocks be modified. Predicted TDG levels associated with a spill of 50,000 ft^3/s as a function of reservoir head and tailwater or submergence are shown in Table 2. This alternative will transfer reservoir TDG levels when the TDG level of the reservoir is greater than that generated by the spill. The TDG level generated by the spill is otherwise combined with reservoir TDG levels as usual. TDG levels generated by spill alone range from 110 percent at reservoir El. 1208 with tailwater El. 985 to 131 percent at reservoir El 1260 with a tailwater El 958. The curves in figures 16 and 17 show TDG levels generated for total river flows associated with reservoir Els of 1208 or 1260 feet with consideration of various possible reservoir TDG levels. The TDG level produced by the 50,000 ft^3/s and the 7Q10 total discharge of 241,000 ft^3/s is 119 percent. This will be used for general comparison of alternative performance. River TDG saturation levels will only be below 110 percent when the lake TDG levels are 105 percent or less.

Table 2. - TDG levels for the extend and cover alternative at reservoir elevation 1208 and 1260 with various lake TDG levels.

Tailwater conditions	Qs cfs	Qpp cfs	Qt cfs	TDG of OW spill %	River TDG for various reservoir TDG levels — TDG Reservoir 100%	105%	110%	115%	120%	125%	130%
Min. tw El. 958	50000	25000	75000	131	120.67	122.33	124.00	125.67	127.33	129.00	130.67
Normal tw range El. 962-966	50000	72000	122000	126	110.66	113.61	116.56	119.51	122.46	125.41	130.00
Max. tw range El. 966-970	50000	118000	168000	123	106.85	110.36	113.87	117.38	120.89	125.00	130.00
Tw El. 970-972	50000	160000	210000	122.5	105.36	109.17	112.98	116.79	120.60	125.00	130.00
7Q10 tw El. 975	50000	191000	241000	119	103.94	107.90	111.87	115.83	120.00	125.00	130.00
Tw El. 980	50000	245000	295000	115	102.54	106.69	110.85	115.00	120.00	125.00	130.00
Tw El. 985	50000	280000	330000	112	101.82	106.06	110.30	115.00	120.00	125.00	130.00
8 blocks operating for mid operation @ reservoir El. 1208											
Min. tw El. 958	50208	24792	75000	126	117.41	119.06	120.71	122.36	124.02	125.67	130.00
Normal tw range El. 962-966	50208	71792	122000	122	109.05	112.00	114.94	117.88	120.82	125.00	130.00
Max. tw range El. 966-970	50208	117792	168000	121	106.28	109.78	113.29	116.79	120.30	125.00	130.00
Tw El. 970-972	50208	159792	210000	119	104.54	108.35	112.15	115.96	120.00	125.00	130.00
7Q10 tw El. 975	50208	190792	241000	114	102.92	106.87	110.83	115.00	120.00	125.00	130.00
Tw El. 980	50208	244792	295000	112	102.04	106.19	110.34	115.00	120.00	125.00	130.00
Tw El. 985	50208	279792	330000	110	101.52	105.76	110.00	115.00	120.00	125.00	130.00
8 blocks operating for mid operation @ reservoir El. 1260											
Min. tw El. 958	62400	12600	75000	131	125.79	126.63	127.47	128.31	129.15	129.99	130.83
Normal tw range El. 962-966	62400	59600	122000	126	113.30	115.74	118.18	120.63	123.07	125.51	130.00
Max. tw range El. 966-970	62400	105600	168000	123	108.54	111.69	114.83	117.97	121.11	125.00	130.00
Tw El. 970-972	62400	147600	210000	122.5	106.69	110.20	113.71	117.23	120.74	125.00	130.00
7Q10 tw El. 975	62400	178600	241000	119	104.92	108.62	112.33	116.04	120.00	125.00	130.00
Tw El. 980	62400	232600	295000	115	103.17	107.12	111.06	115.00	120.00	125.00	130.00
Tw El. 985	62400	267600	330000	112	102.27	106.32	110.38	115.00	120.00	125.00	130.00

Shaded values are below 110 percent TDG supersaturation in the river below Grand Coulee. For definitions, see Table 1.

Construction Features and Cost Estimate

The main construction issues for the extend and cover alternative will be the excavation and removal of concrete from the downstream face of the dam and the cofferdam. Mechanical excavation of the concrete would be feasible, but with the large extent and volume of material, it will be time consuming and wire saw cutting is not viewed as practicable at this time. The contractor will probably want to use blasting to increase production rates. However, with the nearest outlets less than 100 feet from the right powerplant, the issue of blasting will need to be reviewed in greater detail prior to its use as the approved method of excavation. Debris falling and damaging concrete in the roller bucket and removal of concrete to an approved disposal site are other concerns.

The cofferdam for this alternative must withstand a differential head of up to 100 feet. The base of this cofferdam will rest in the concrete roller bucket and any damage of the roller bucket will be repaired. The sealing of the cofferdam to the existing concrete may require a fairly elaborate dewatering system during construction.

The design of the cofferdam will impact the size of the cranes required for delivering material. The slope of the face is at 0.8:1 and with a 100-foot-tall cofferdam a crane will have to have a reach of about 80 to 100 feet loaded with material and supplies to reach the farthest point on the face of the dam. Work will be accomplished using barges and cranes.

Typically, cofferdams are the property of the contractor and that was assumed in the estimates. Construction on the downstream face of the dam and the flip bucket area will require extensive use of cofferdams. The cofferdam will be constructed similar to a bulkhead and anchored off the face of the dam. The bulkhead will need to be about 100 feet tall and about 50 feet wide (to cover one block). The estimates provided assume the use of one cofferdam if one or two blocks are being modified, otherwise the estimate assumed two bulkheads to facilitate the overall construction.

The construction duration for modifying eight blocks with the extend and cover alternative is estimated at 3 years. The use of two cofferdams reduces the time of construction from 48 months to 36 months for the design flow of 50,000 ft^3/s or eight blocks. This is based on six months of work per block, with work on two blocks at the same time but offset by four months and some time for mobilization and demobilization. Unknowns with cofferdam construction, potential work area limitation, constricted access, and the disposal location could negatively impact costs in the final design stage. No loss of powerplant generation capability or revenue is anticipated.

The details of the listed items and costs are shown in appendix 1 for this estimate and summarized in Table 3. Cost estimates for any variation in number of blocks from one to all ten are shown with the field cost ranging from $14,000,000 to $79,000,000 (see Table 3). The field cost for the design alternative with 50,000 ft^3/s capacity is estimated at $74,000,000. The main design difference between the feasibility and conceptual designs is the use of the diffuser for energy dissipation and turbulence reduction at the end of the extension and the reduction from nine to eight pairs of outlets. The cost difference is primarily a function of estimate refinement. The PN Region requested that non-contract costs of 30 percent be added to the field cost for a closer evaluation of total costs. The non-contract costs are design costs, construction management, etc. The non-contract cost for alternative 1 using a design flow of 50,000 ft^3/s and modifying eight blocks would be $22,200,000. The total cost for the design and construction of the extend and cover alternative for the 50,000 ft^3/s design flow is estimated at $96,200,000.

Table 3. - Cost ($ millions) for design and construction of the extend and cover the outlet works alternative with a diffuser at the end for various numbers of outlet pairs or blocks modified (Alternative 1).

Number of Blocks Modified	**1 Block**	**2 Blocks**	**3 Blocks**	**4 Blocks**	**5 Blocks**	**6 Blocks**	**7 Blocks**	**8 Blocks**	**9 Blocks**	**10 Blocks**
Extend outlet works behind the face of Spillway	**$14**	**$27.5**	**$38**	**$48**	**$56**	**$63**	**$68**	**$74**	**$77**	**$79**
Non Contract Costs	**$4.2**	**$8.3**	**$11.4**	**$14.4**	**$16.8**	**$19**	**$20.5**	**$22.2**	**$23**	**$24**
Total Cost	**$18.2**	**$35.8**	**$49.4**	**$62.4**	**$72.8**	**$82**	**$88.5**	**$96.2**	**$100**	**$103**
Q (min head, El. 1208)	**6276**	**12552**	**18828**	**25104**	**31380**	**37656**	**43932**	**50208**	**56484**	**62760**
Q (max head, El. 1260)	**7800**	**15600**	**23400**	**31200**	**39000**	**46800**	**54600**	**62400**	**70200**	**78000**

The table of cost comparisons for the extend and cover alternative will also be used for cost comparisons for joint operation possibilities.

Deflectors - Minimal Number of Outlets (Alternative 3)

Description

The objective of this alternative was to reduce TDG production. Adding deflectors on the downstream face of the spillway below the outlet works, figure 18, would reduce the TDG levels that are currently experienced in the tailwater, but not to the levels of the TDG concentrations in the reservoir. The increase in TDG generated by outlet use will be reduced by constructing deflectors below the outlet works to prevent the plunging of the releases to the invert of the flip bucket. The conceptual level design was constructed and tested in the laboratory using the 1:17 model of the paired outlet works to evaluate and refine deflector options.

The outlet works control gates and the exit point on the downstream face of the dam will not be modified. Therefore, the discharges will match those currently experienced. The fully open mid-level outlets will each deliver approximately 4480 ft^3/s when the reservoir is at El 1208 or the level of minimum drafting. To accommodate a total spill of 50,000 ft^3/s, with no modification to the 102-inch ring seal gates, a total of 12 outlets will be utilized, and a minimum of six blocks will need to be modified.

The deflector design has a reverse radius of approximately 70 feet with a horizontal extension of about 6 feet for a total deflector length, out from the dam, of approximately 30 feet. Based on the results of the model study, the deflector should be located with the horizontal section at El 965, figure 18. All deflectors would be constructed at the same elevation.

The discharge from the mid-level outlets ranges from approximately 4,480 to 5,350 ft^3/s per outlet, depending on the reservoir water surface elevation. With two outlets per block, the total discharge per block ranges from 8,970 ft^3/s to 10,700 ft^3/s. This discharge spreads over the 50 foot block width. In the conceptual phase it was assumed that flow spreading and structural support would require continuous placement of the deflectors across intermediate blocks. Based upon model results and construction considerations, the modification will be made per block to only those blocks beneath the outlets.

Evacuation concerns were reviewed for this design and will be evaluated further in the final design. The evacuation for this alternative remains the same as currently. The evacuation of FDR Lake is met if the powerplant outlets are included in the evacuation.

A 65-ft-high cofferdam will be built by forming for the defector inside the cofferdam. There will be unwatering and dewatering due to cofferdam leakage.

This alternative can be modified to accommodate a larger or smaller design discharge relatively easily. The discharge limitations range from a minimum of one pair of outlets in one block for a discharge of 8,970 ft^3/s, to all ten pairs of outlets in 10 blocks for a discharge of 89,700 ft^3/s.

Hydraulic and Total Dissolved Gas Evaluation

Hydraulic Analysis

The sectional 1:17 outlet works model was used to determine the deflector geometry and location, observe flow conditions, and qualitatively evaluate the gas transfer characteristics for this alternative. This alternative places deflectors below the mid-level outlets and assumes that only the mid-level outlets would be used to pass the design flow. The deflectors must be designed for operation of the mid-level outlets from reservoir El 1208 to 1260 and tailwater range from El 958 to 975 for the 7Q10 event. However, the deflector must also perform adequately under other operating scenarios including operation of the upper-level outlet works only; combined operation of the upper- and mid-level outlet works; and spillway operation. Hydraulically, the deflector under various operating scenarios should turn the flow downstream and prevent plunging while the outlets or spillway are operating. Structurally, the deflector must withstand the additional loading produced by the jet impingement under all feasible operating conditions.

Flow from the outlets at Grand Coulee produces significantly different flow conditions for deflector design than flow conditions that occur on COE gated spillways [7]. The typical COE deflector is designed with flow attached to the spillway face. The flow profile has a smooth transition from the spillway face to the horizontal elevation of the deflector and the deflector fully supports the flow throughout the trajectory.

The available head at Grand Coulee produces higher velocity jets than typically occur at COE sites where deflectors have been installed. The outlets exit the face of the dam with an angle 10 degrees flatter than the dam slope. This difference in angle with the high velocity at the exit produces a jet trajectory that springs free from the dam face. The jet then impinges directly on the deflector where the jet spreads laterally and is directed downstream. The jet trajectories and impact locations on the deflector vary with the use of the upper, mid, or combined outlets and with reservoir elevation. Unit discharges are 180 and 214 ft^3/s per linear foot for reservoir elevations ranging from 1208 to 1290 feet for jet spread over the 50 foot width of the deflector. Typical jet trajectories for mid-level outlets and the resulting deflector impingement for reservoir El. 1208 and 1290 and minimum tailwater are shown in figures 19 and 20. Note that with higher head the impact location moved further out toward the downstream edge of the deflector block.

The modeled deflector initially included a 50.0 foot radius and a 6.0 foot long horizontal flat section. For the initial model study, the observed deflector was positioned below the mid-level outlet with the flat section at El 951.0 feet. This elevation was 5.0 feet below the minimum tailwater elevation of 956.0 (the tailwater elevation ranges from 956.0 to 975.0). The model tailbox was sufficiently long that the extent did not distort plunging characteristics of the deflected jet. Model observations of the jet characteristics under various tailwater elevations set the final elevation of the horizontal section of the deflector at El 965. The radius was changed to 70 feet for final design. This radius helped reduce backflow caused by impingement on the deflector.

The outlet works deflector flow conditions investigated were the mid-level outlets only; the upper-level outlets only; and the combined mid- and upper-level outlets with the full range of tailwater levels. Figures 19 through 30 show various flow conditions under the different operating conditions. High velocities and jet impact on the deflector produced break up of the jet across the deflector, as opposed to typical deflector installations. Jet impact on the deflector also caused pronounced surface turbulence for all outlet works operating conditions.

Figures 19 and 20 show mid-level outlet works operation with low tailwater and with minimum and maximum reservoir head. The close proximity of the deflector to the outlet discharge produces less impact than with other operating conditions. The jet moves outward as the reservoir elevation and jet velocity increases. At the tailwater elevation below the deflector the jet leaving the deflector fans out and is almost flat. The jet does not plunge deeply into the tailwater.

Figures 21 through 24 show a series of top and side views of the operation for the upper-level outlets only with the tailwater below the deflector. The jets from the upper outlets impact on the deflector very close to the face of the dam with some residual lateral spread falling over the edge of the deflector for reservoir El 1208, figures 21 and 22. The jets from the upper outlets seemed to converge to the centerline between the outlets then fan out over the edges. Under reservoir El 1290, the jets from the upper outlets clearly spread laterally and downstream prior to impact on the deflector, figure 23. Residual flow actually overshoots the downstream edge of the deflector, figure 24.

The top and side views for the combined upper- and mid-level outlet works and tailwater El 958 below the deflector are shown in figures 25 through 30. Just as the flow condition indicates, the jets for the combined operation impact together on the deflector. The mid-level jets support the upper level jets preventing over flow of the upper jets and directing the flow downstream and laterally in an efficient manner, figures 25 and 27. The combined discharge is large compared to either outlet level alone, but the flow conditions are improved over the upper outlets operating alone. Surface spray, turbulence intensity, and velocities are increased with the combined operation, but plunge depths are not significantly different, figures 26 and 28. Maximum reservoir El 1290 particularly produces an excessive amount of spray. Figures 29 and 30 show combined outlet operation under reservoir El 1290 and tailwater El 970. Under this condition, the jet from the deflector begins riding up on the tailwater and the downstream jet becomes more confined by the surrounding tailwater. Unfortunately, the side views of the deflector operation include unnatural plunging of bubbles down the side wall of the model after impact of the spreading jet on the wall. This portrays an improper view of the depth of the recirculation expected in the prototype.

The model only included one outlet pair. The adjacent outlet pair would be discharging parallel to this pair about the location of the model wall. Effect of multiple outlet pairs operating could not be determined by the model but most likely would not significantly effect the overall flow conditions.

Figures 19 through 30 show the final design and elevation of the deflector over the expected range of tailwater elevations. With traditional deflectors, the curvature of the deflector smoothly turns the flow horizontally and in the downstream direction without creating the high turbulence levels experienced here. These figures have shown that the design of the deflector radius was of little importance to outlet operation because the jet impacts downstream from the curvature. The radius was needed for spillway flow and cavitation considerations discussed in later sections. The elevation chosen for the deflector warrants further discussion and is based upon visual observations of the jet interaction with the tailwater. Traditional tailwater interactions below a deflector that vary from flow ride-up with resulting plunging, to skimming, to direct plunging as a function of tailwater elevation were not observed [7].

The mid-level outlets were operated over a wide range of tailwater elevations above and below the deflector elevation. For tailwater elevations lower than the deflector, the high velocity jet impacted on the deflector, dispersed, and broke-up as it was deflected at a flat trajectory to the tailwater. The resulting plunging was shallow. These conditions appeared to be constant for tailwater elevations down to approximately El 956 or the minimum possible tailwater. With a tailwater somewhat above the deflector, there is a surface circulation pattern that tends to entrain air and supersaturate the water. At maximum tailwater on the deflectors, plunging was observed with energy dissipation directly over the deflector. Even with this condition a direct horizontal deflection of the jet across the tailwater was present. These observations indicated that the elevation of the lip of the deflector should be set at a higher rather than lower elevation within the expected tailwater range. This balances the resulting surface turbulence against the re-entrainment of gases and re-saturation of the water downstream from the deflectors.

Velocities were measured in the model about 185 ft downstream from the end of the flip bucket. Velocities ranged from about 15 to 45 ft/s near the surface of the tailwater for all ranges of flow conditions and tailwater elevations studied, figure 31. Downstream energy levels in the skimming jet will be higher than currently experienced in the tailrace channel. This may impact and degrade the bank protection downstream from Grand Coulee Dam and should be evaluated using a 3-dimensional physical model in final design.

A shear zone will be created by the high velocity jet traveling along the surface of the tailwater off the deflectors. A reverse roller forms underneath the surface jet that could potentially include high velocities. Whether or not this is a disadvantage will depend upon the availability of material that can be drawn back into the roller bucket at the base of the spillway with operation of the deflectors. This problem has been documented at Reclamation's Yellowtail Afterbay Dam when metal deflectors were added; however, the deep basin at Grand Coulee Dam likely reduces this concern. Because of model extent limitations this could not be addressed.

Simulated Spillway Flows

A condition that simulated the spillway operation with a unit discharge approaching 600 $ft^3/s/ft$ was evaluated to observe the behavior of the deflector under spillway flows. Spillway flows were simulated by constructing a pressure box over the top of the pair of upper outlets. The pair of upper outlets has a capacity of 8,520 ft^3/s under reservoir El 1290. This is a unit discharge of 374.5 $ft^3/s/ft$ given the diameter of the two outlets and the spacing between outlets of 22.75 feet. The pressure box was constructed with an adjustable downstream opening to produce the flow depth associated with a unit discharge of 600 $ft^3/s/ft$ and a corresponding velocity from the spillway. Figure 32 shows the simulated spillway flow attached to the face of the dam and impacting on the deflector. This large spillway flow produced less impact on the deflector than outlet operation.

The use of a smooth curve to transition the flow over the deflector was needed for this flow condition to reduce turbulence. A smaller radius for the deflector shape was not acceptable because the simulated spillway flow was not turned by the smaller radius, but impinged on the flatter sections and dispersed turbulently. The radius for the deflector was determined by an evaluation of the desired hydraulic behavior of the jet from both the outlet works and the spillway flows. This simple investigation somewhat alleviated concerns regarding spillway flow over the deflectors.

Pressure Results

Static pressure data were initially recorded using piezometer taps and a manometer board to map the general tendencies of the jet impingement and to locate the areas to place dynamic pressure cells. Static pressures were measured in twenty four locations on the centerline between the two outlets, on the centerline of the left outlet, and all along the downstream edge of the deflector. Dynamic pressure cells were located one each on the centerline between the outlets and on the outlet centerline at the locations of the maximum static impingement. Mapping of these pressures also indicated the flow conditions and confirmed the jet trajectory observations.

Figures 33 and 34 show the static and dynamic pressures on the deflector with combined upper- and mid-level operation. Figure 33 is the record of the pressures as measured at the centerline of the deflector, midway between the outlets. Figure 34 is the record of the pressures for the combined flow at the center of the left outlet. The highest recorded pressure, about 138 feet, occurred with a reservoir El. 1290 on the centerline between the two outlets about 14 to 18 feet out on the deflector from the face of the dam. Pressures occurring for reservoir El 1208 were often recorded at the level of the tailwater or simply atmospheric as the water from the jets missed the measurement locations entirely. In the later case, the pressure loading would be negligible. A maximum differential pressure of 120 feet was used for the structural loading analysis and design of the deflectors to reflect an average maximum pressure over the entire deflector. The flow rate for the combined operation is obviously considerably higher than with either outlet operating alone, and the overall pressure profiles reflect greater average impact and flow depths on the deflector. However, the maximum pressure did not exceed the upper outlets alone and was 102 feet with reservoir El 1260. The maximum pressure was measured at about 14 to 18 feet from the face of the dam. The trends were identical to those recorded with the mid-level outlet except for the impact on the centerline between the outlets was high further out from the face. The mid-level outlet operation helped support the upper outlet flow preventing impingement beyond the end of the deflector and moderating the impact.

Impact pressures measured with a simulated spillway unit discharge of 374.5 $ft^3/s/ft$ or total flow of 630,000 ft^3/s and reservoir El 1290 were less than those observed with the combined outlet works discharge onto the deflector. Therefore, the deflector should withstand spillway flows without failure if the deflector is designed for the outlet works jet pressures.

Cavitation Analysis

The final shape and radius of the top surface of the deflector was determined based upon ease of construction, cavitation, and jet behavior at impact. The 70 foot radius improved dispersion of the flow from the outlet works and was also needed for proper spillway flow conditions. Cavitation is not a concern for the outlet works operation because the jets are not attached to the deflector surfaces. A cavitation analysis [8] was performed on the incremental discharges for the spillway operation, because the deflectors will split the flow diverting some onto the deflector and carrying some flow on the existing spillway surface. Also, the deflector surfaces will alternate between being submerged and not. The cavitation analysis was performed using the cavitation index, ó, which expresses the cavitation potential as a function of the difference between a reference pressure and vapor pressure divided by the product of the velocity head and flow density. The cavitation analysis produced cavitation indices of 0.7 to 0.45 for unit discharges ranging from 50 to 611 ft^3/s per linear foot. Because cavitation damage does not usually occur until the index drops to 0.2 it would normally not be considered a problem [8]. However, since the velocities also reach 68 to 73 ft/s for the unit discharges ranging from 50 ft^3/s per linear foot to the PMF condition of 611 ft^3/s per linear foot, a little more thought was given to the deflector shape. With spillway operation the deflector will be similar to a chute block at the entrance to a USBR Type II basin. At the PMF the flow depth at the deflector location is about 12 feet. The deflector thus will have a major influence on the flow. Incipient cavitation begins when the cavitation index is 0.18. This could occur on the face of the spillway. The deflector is too large to treat as an offset into the flow, but experience has shown [p.32 of 8] that damage observed in hydraulic structures occurs downstream of an irregularity at distances up to 100 times the height of the offset. Thus the cavitation index of the flow must be on the order of 1/6 of the incipient cavitation index or 0.18/6= 0.03. The cavitation indexes for Grand Coulee Dam spillway flows are at or above this and in the range of decreasing damage rates. Therefore, cavitation damage is not expected on the surface of the deflectors.

The edges and sides of the deflector were still of concern. Investigation into stilling basins and slotted flip buckets shows that "teeth" have successfully operated in these environments for velocities from 38 to 108 ft/s for a Type II basin and for velocities up to 75 ft/s for a slotted bucket. The "teeth" on a slotted bucket are rounded. The edges of the deflector will also be rounded to prevent or limit initiation of cavitation.

Lastly, the flow rate from the spillway will generally be of the range of smaller unit discharges that will more than likely produce highly aerated flow, which is not likely to cause cavitation.

Structural Analysis

The structural analysis on the deflector was performed using the loading information developed from the hydraulic model study. The impact on the deflector was determined for static loading using both piezometer taps and manometer boards and for dynamic loading using dynamic pressure cells to provide information on the loading fluctuations.

ACI-318 (99) [9] for the design of corbels and shear friction were used to structurally design the deflectors. The concrete was 4000 lbs/in^2 at 28 days and the steel has a yield strength of 60,000 lbs/in^2.

The structural design was developed based upon the dynamic pressure data that was recorded using a laptop computer and software. The software statistically reduced each data set. Pressure data were recorded for reservoir Els 1208, 1260, and 1290 with upper-, mid- and combined-outlet operation. Tailwater was set at El. 958 for the measurements, because the static pressure results had shown that the tailwater above the deflector only caused an increase in static pressure equal to the submergence depth. The structural design was based upon the combined flow condition under reservoir El 1260, given that spillway releases will be used above El. 1260. Observed mid-level outlet pressures were always below the combined outlet pressures. Observed upper-level outlet pressures were similar to the combined pressures because of the higher energy in the jet falling from the higher elevation. The combined pressures used in the structural design for loading on the deflector are shown in figures 33 and 34. As previously discussed, the structural loading was based upon a uniform pressure head of 120 feet at maximum reservoir water surface El. 1260.

An analysis of the hydraulic flow profiles was undertaken to determine the radius of the deflector. The analysis determined the water surface profiles, velocities, and depths down the dam face to the various tailwater elevations under consideration. Because the outlet flows were detached from the spillway face and not guided by the deflector flow surface, only spillway flows were studied. The analysis for the deflector alternative 3 used the water surface profile program, CTAC. The spillway discharge with a maximum of 611 ft^3/s per linear foot was assumed for designing the radius of the deflector bucket.

The velocity for the spillway flow of 611 ft^3/s per linear foot was computed to be 73 ft/s at the deflector at El 965. This velocity was used to compute the radius of curvature needed for the spillway flow and compared well with the radius of the deflectors. The following analysis was performed to confirm the acceptability of the 70-foot radius used in the model study. It was performed to ensure that a smooth transition would be provided between the dam face and the deflector given the high velocities. The radius of curvature for the deflectors was determined from the following static equation for flip bucket design from Design of Small Dams [10], page 385:

$$p = \frac{2qv}{R} \tag{4}$$

where p = dynamic pressures (lbs/ft^2) (assuming 1000 lbs/ft^2 if no model test were performed for the spillway and p = 112 feet or 7000 lbs/ft^2 for the combined outlet works operation), q = unit discharge (ft^3/s/ft), v = velocity (ft/s), R = radius (ft)

However, since no pressure data was taken for the spillway operation, an approximate range of radii for the deflector alternatives were determined using a dynamic pressure of 1000 lbs/ft^2. A calculated 89-foot radius was required to turn the flow (at 611 ft^3/s/ft, 73 ft/s). The model was tested with a 68-foot radius and was acceptable for the spillway and outlet works flows. A larger radius (70 foot versus 68 foot) is more desirable since it provides for larger spillway discharges and has more effect in guiding the jets from the outlet works in a downstream direction. The shorter and flatter radii aren't as effective in guiding the jets downstream from the outlet works and this results in flow being spread out in all directions rather than in a downstream direction.

Total Dissolved Gas Evaluation

The 50-ft-wide deflector was positioned below the mid-level outlet with the horizontal section at El. 965 ft. This elevation is 7.0 ft (prototype) above and below the minimum and maximum normal tailwater elevation range of 958 to 972. The modeled deflector geometry includes a 70 ft radius and a 6.0 ft long horizontal section for a total deflector length out from the dam face of 30 ft.

The outlets are operated fully open or fully closed because the ring seal gates at the inlet are not to be used for control. Therefore, variations in unit discharge are limited to the available head influences only. The length of the tailbox did not distort plunging characteristics of the deflected jet.

Mean bubble plunge depth created by the skimming jet near the end of the tailbox as a function of tailwater elevation were used as rating parameters for the TDG analysis.

Observed bubble entrainment depths in the tailwater pool were fairly constant for tailwater elevations ranging from approximately El. 972 to approximately El. 956. Entrainment depths appeared to be a function of the shear between the deflected jet and the tailwater pool. Since energy levels were fairly constant in the jet over this tailwater range, the depth of penetration of vertical eddies driven by the shear also appeared to be fairly constant. A rooster tail formed as the tailwater increased above the level of the deflector until the deflector was fully submerged by the tailwater. Plunging generated by the rooster tail did not appear be significant. Substantial potential for gas production is evidenced by the presence of air bubbles to an average depth of 15 feet.

Velocities developed downstream from the deflector with mid-level operation are shown in figure 31 for tailwater Els 958 and 975. This is a slightly higher tailwater elevation than "normal" but will allow comparison and prediction of velocities as the tailwater rises. Velocities produced by operation of the mid-level outlets with either tailwater El 975 or 958 ranged from about 5 ft/s at a depth of about 15 ft to about 45 ft/s at the surface. The vertical extent of the high velocity flow seem to be very similar for all evaluated conditions with about a 15 ft depth limit. The difference in tailwater produced minimal difference in the measured velocities. The higher tailwater actually produced slightly higher overall velocities downstream caused by lateral constriction of the jets by the tailwater with insufficient tailwater to actively dissipate energy by hydraulic jump formation.

Combined flow velocities are also shown on figure 31. As may be seen, the flow velocities obtained for combined operation are almost the same as those obtained for mid-level outlet operation only. Velocities with tailwater El. 975 are also slightly higher than velocities with tailwater El. 958. The flow is again confined laterally and rides up on the tailwater downstream of the deflector. The combined flow velocities again ranged from 5 ft/s at a depth of 15 ft to about 45 ft/s at the surface.

These velocity measurements may be used to help quantify the expected TDG levels and tailwater degassing generated by deflected flow as they verify the visual observation of a mean bubble depth of about 15 ft. It is expected that the entrainment depth, thus TDG predictions, are conservative due to potential off-gassing that could occur in the prototype.

Figures 35 and 36 show the TDG levels predicted for the deflectors operating over the full range of flows and tailwaters. The mean bubble depth was used to rate the TDG generation capability of the deflectors. It became obvious that the impingement on the deflector spread and dispersed the flow such that the generated bubble entrainment depth was fairly constant throughout the flow range of the upper outlets only, the mid outlets only, and combined flows. The deflector was placed midway between the expected normal operating tailwater range which produced free skimming jet flow for minimum tailwater levels and a partially formed hydraulic jump for high tailwater levels. The bubble plume did not plunge more than 6 ft below the level of the deflector as the tailwater increased. The depth of the bubble plume did not vary with head or discharge but only with tailwater relationship to the deflector elevation.

The predicted TDG level for the 50,000 ft^3/s design discharge varied from 124 percent at tailwater El. 958 to 136 percent at tailwater El. 985. Over the full reservoir range from El. 1208 to 1290 and the “normal” tailwater range of 958 to 970, the TDG levels off the deflector were estimated to range from approximately 124 to 126 percent depending on the various factors. Six blocks are needed to pass the design flow rate under reservoir El 1208. The curves in figures 35 and 36 show how the total river TDG levels will vary with mixed flow using the various lake TDG levels. The TDG saturation levels will only be 110 percent or less when the lake TDG levels are at 105 percent or less.

Table 4. - TDG levels for the deflector alternative under the 7Q10 event at reservoir elevation 1208 and 1260 with various lake TDG levels.

	Qs	Qpp	Qt	TDG of OW spill	TDG Reservoir 100%	105%	110%	115%	120%	125%	130%
	cfs	cfs	cfs	%	River TDG for various reservoir TDG levels						
Min. tw El. 958	50000	25000	75000	124	116.00	117.67	119.33	121.00	122.67	124.33	126.00
Normal tw range El. 962-966	50000	72000	122000	124	109.84	112.79	115.74	118.69	121.64	124.59	127.54
Max. tw range El. 966-970	50000	118000	168000	125	107.44	110.95	114.46	117.98	121.49	125.00	128.51
Tw El. 970-972	50000	160000	210000	126	106.19	110.00	113.81	117.62	121.43	125.24	129.05
7Q10 tw El. 975	50000	191000	241000	128	105.81	109.77	113.73	117.70	121.66	125.62	129.59
Tw El. 980	50000	245000	295000	132	105.42	109.58	113.73	117.88	122.03	126.19	130.34
Tw El. 985	50000	280000	330000	136	105.45	109.70	113.94	118.18	122.42	126.67	130.91
6 blocks operating for mid operation @ reservoir El. 1208											
Min. tw El. 958	53800	21200	75000	124	117.22	118.63	120.04	121.46	122.87	124.28	125.70
Normal tw range El. 962-966	53800	68200	122000	124	110.58	113.38	116.17	118.97	121.76	124.56	127.35
Max. tw range El. 966-970	53800	114200	168000	125	108.01	111.40	114.80	118.20	121.60	125.00	128.40
Tw El. 970-972	53800	156200	210000	126	106.66	110.38	114.10	117.82	121.54	125.26	128.98
7Q10 tw El. 975	53800	187200	241000	128	106.25	110.13	114.02	117.90	121.79	125.67	129.55
Tw El. 980	53800	241200	295000	132	105.84	109.92	114.01	118.10	122.19	126.28	130.36
Tw El. 985	53800	276200	330000	136	105.87	110.05	114.24	118.42	122.61	126.79	130.98
6 blocks operating for mid operation @El. 1290											
Min. tw El. 958	64000	11000	75000	124	120.48	121.21	121.95	122.68	123.41	124.15	124.88
Normal tw range El. 962-966	64000	58000	122000	124	112.59	114.97	117.34	119.72	122.10	124.48	126.85
Max. tw range El. 966-970	64000	104000	168000	125	109.52	112.62	115.71	118.81	121.90	125.00	128.10
Tw El. 970-972	64000	146000	210000	126	107.92	111.40	114.88	118.35	121.83	125.30	128.78
7Q10 tw El. 975	64000	177000	241000	128	107.44	111.11	114.78	118.45	122.12	125.80	129.47
Tw El. 980	64000	231000	295000	132	106.94	110.86	114.77	118.69	122.60	126.52	130.43
Tw El. 985	64000	266000	330000	136	106.98	111.01	115.04	119.07	123.10	127.13	131.16

Shaded values are below 110 percent TDG supersaturation in the river below Grand Coulee.

Construction Features and Cost Estimate

The deflector alternative 3 is expected to require 3 years for completion of construction of 6 deflectors assuming work on a single block at a time. About 3 months will be needed to construct a deflector on each of the 6 blocks being modified on the dam. It is estimated that an average of 1 ½ months will be required to setup and remove the cofferdam from the block. The work can be accelerated by working on multiple blocks at a time. The original construction time was estimated at 2 years based on the COE experience at their dams with construction on 5 blocks. Due to the increased number of blocks (versus 5 blocks), larger size of the deflector, and the complexity of the work and cofferdam, the duration of construction was lengthened by 1 year.

The construction of deflectors applies proven technology developed by the COE at dams on the Lower Snake and Lower Columbia River in Washington. The design for the deflectors at Grand Coulee Dam is more complicated because the design unit discharges and subsequent forces on the deflector are higher. Furthermore, the operating head is considerably larger and the behavior of the jets from the outlet works are different than these previous designs.

Some of the unresolved construction issues include the cofferdam, dewatering, and forming scheme for the deflectors. Costs for the conceptual estimate were based on a separate cofferdam and forming system. It may be possible to incorporate the cofferdam and form into one to save costs.

The estimate assumes a separate cofferdam, 65-ft-high, will be built with forming for the deflector inside the cofferdam. A system to dewater the work area will be required due to leakage past the cofferdam. An alternative cofferdam was constructed as part of the downstream form for the deflectors at the Corps of Engineers (COE) John Day Dam [11]. The COE design will reduce costs but was not included for this estimate. Construction on the downstream face of the dam will require extensive use of cofferdams. There is no loss of powerplant production assumed for this alternative during construction.

The cofferdam will extend from El. 980 to El. 915, or will be 65 ft tall. Dewatering may be a significant issue because the cofferdam will have an unbalanced head or water pressure of about 55 feet. The length of the cofferdam will be 50 feet or the block width plus the triangular end sections.

Preparation of concrete for this alternative may also be difficult because of requirements to anchor the deflectors to the face of the dam, and because of the required surface preparation on the face. This surface preparation work was estimated as a mechanical process with equipment.

The detailed items and costs are shown in appendix 2 and summarized in Table 5. The cost estimate was prepared assuming a pair of outlets or 1 block was modified and then multiplied by the number of blocks needed to support the deflectors. This option can be easily modified to accommodate a larger or smaller design flow. The field cost for this alternative for a design flow of 50,000 ft^3/s is estimated at $12,000,000. The major design difference between this estimate and the previous study is the total volume of material was nearly cut in half by eliminating the deflectors on the blocks between the outlets. The unit cost estimates of the concrete excavation on the dam face and the barge rentals for this estimate are higher than previous estimates. The total cost for a 50,000 ft^3/s design flow includes a 30 percent non-contract cost of $3,600,000 for a total estimated job cost of $15,600,000.

Table 5.- Cost ($ millions) for deflectors with a 70 foot radius on the face of spillway under the outlet works.

No. of Blocks Modified	1 Block	2 Blocks	3 Blocks	4 Blocks	5 Blocks	6 Blocks	7 Blocks	8 Blocks	9 Blocks	10 Blocks
70-foot radius Deflectors on the face of Spillway	$3	$5	$6.9	$8.7	$10.5	$12	$13.5	$15	$16.5	$18
Non-contract cost (30%)	$1	$1.5	$2.1	$2.6	$3.1	$3.6	$4	$4.5	$5	$5.5
Total Cost	$4	$6.5	$9	$11.3	$13.6	$15.6	$17.5	$19.5	$21.5	$23.5
Q (min head, El. 1208)	8970	17900	26900	35900	44900	53800	62800	71800	80700	89700
Q (max head, El. 1260)	10060	20120	30180	40240	50300	60360	70420	80480	90540	100600

Cost estimates have not been prepared for potential effects on the river bank stabilization, which may be significant. It is estimated that there may be costs associated with required improvements especially in the area between the dam and the bridge below the dam. The table of cost comparisons for the extend and cover alternative will also be used for cost comparisons for joint operation possibilities.

Forebay Pipe with Cascade (Alternative 5)

Description

The objective of this alternative was to reduce the TDG levels in the reservoir by constructing a totally new cascade structure adjacent to the Third Powerplant on the north bank of the river. The design will transport the design flow of 50,000 ft^3/s to the river through a large gated tunnel constructed at the end of the forebay dam. The tunnel will discharge into a manifold gate chamber, stilling well, stripping cascade and stilling basin, figures 37 to 40. The TDG objective was to produce downstream TDG levels no greater than the water quality standard of 110 percent. Critical to the design is the ability of the stilling well and cascade to dissipate energy and strip gas and of the stilling basin to prevent plunging of the resulting flow when returned to the tailwater pool.

The manifold gate chamber, gate type, and cascade slope were modified from the original concept phase during the design of the 1:43.68 Froude-scale hydraulic model. The initial model design is shown in figure 41 operating under a discharge of 50,000 ft^3/s with reservoir El. 1290.

The model study results indicated that modifications to the conceptual design were needed to improve the performance of this alternative. The changes included the construction of a new pipeline, clamshell gates, manifold gate chamber, stilling well, baffled cascade, and vertically adjustable stilling basin. These elements were all modified during the model study.

The new pipeline will be constructed in the area of the right abutment, extending from the end of the existing forebay dam to the current north service yard for the Third Powerplant. There, the gate chamber, stilling well, baffled cascade, and stilling basin will be constructed (figures 37 to 40.) The work requires removal of the existing end wall of the forebay dam and construction of a new end wall as a gravity dam with the same cross section as the existing forebay dam. The end wall will include a trashrack on the upstream face and contain a wheel-mounted guard gate with air vent for a 40-ft-diameter tunnel through the right abutment. The tunnel, gate chamber, stilling well, baffled cascade, and stilling basin will require excavation. The gate chamber will be an 820-ft-wide manifold structure, housing sixteen 7-foot 6-inch-diameter clamshell gates that will discharge into a 50-ft-long stilling well. The flow in the stilling well will be directed down by a fillet attached to the downstream wall the entire width of the well. The downstream wall of the stilling well forms a broad crested weir at the top of the 2:1 sloping baffled cascade. The vertically adjustable stilling basin will be located at the toe of the 2:1 sloping cascade. Additional baffle blocks will be located on the stilling basin floor. The basin was designed to accommodate the fluctuating tailwater elevations and prevent re-plunging of the flow. This will be accomplished by designing the basin floor to raise and/or lower as the tailwater fluctuates, keeping the floor of the basin no more than five feet below the tailwater surface. This will be accomplished by stabilizing the stilling basin on piers and using hoists, the buoyant weight of the stilling basin, and an air bladder to float the basin up or down.

The design has changed significantly from the conceptual design. The 39 guard gates from the conceptual design were eliminated for cost reduction. The design now uses the wheel mounted gate as a guard gate for dewatering the gate-chamber manifold. Energy dissipating orifice rings were included in the tunnel to break head but the velocities were still too high for the concept proposed high pressure butterfly valves. The control valves were changed to clamshell gates which are better suited to controlling high velocity flow. The diffuser covering the stilling well was found to be unnecessary. The stepped cascade was replaced by a baffled cascade. The vertically adjustable stilling basin replaces the 6:1 stepped apron and limits tailwater depths to five feet.

Mass and conventional reinforced concrete will be used to construct this alternative. The mass concrete is used to construct the end wall at the forebay dam and conventional reinforced concrete is used in the tunnel, the manifold structure, the stilling well, the baffles on the 2:1 slope, and the adjustable stilling basin. The baffled cascade will be built using mass concrete at a 2 (H) to 1 (V) slope with nine rows of baffle blocks.

Mechanical

The mechanical items included in this alternative include 16 7.5-foot-diameter clamshell gates for controlling flow into the tunnel, one wheel mounted gate to isolate the system for repair and maintenance, 48 hoists needed to stabilize and manipulate the adjustable stilling basin floor, the air supply equipment for the adjustable stilling basin floor, ventilation equipment, and an elevator.

The wheel-mounted gate is located on the forebay dam extension. It will be operated from the existing equipment located on the crest of the forebay dam. This existing equipment is currently used to operate the existing wheel mounted gates of the Third Powerplant. The clamshell gates are located underground in a continuous gate chamber running the length of the manifold. The gate chamber design includes furnishing and installing a ventilation system consisting of a 4000 ft^3/m centrifugal fan, 100 feet of 18-inch diameter schedule 10 carbon steel pipe, and 660 feet of 18-inch diameter schedule 40 PVC pipe in the gate chamber.

There are 48 hoists located outside and above the adjustable stilling basin on top of the 48 piers in the stilling basin. These hoists are needed to help stabilize the stilling basin floor, prevent racking of the stilling basin floor panels, and move the stilling basin as the tailwater surface moves. The air bladders are located underwater, below the stilling basin floor and are needed to displace water and provide buoyancy to help move the stilling basin floor.

The controls for the clamshell gates, the air supply equipment, and the hoist equipment will be located in the Third Powerplant. Auxiliary equipment for all the controls will also be located in the continuous gate chamber of the manifold.

Access to the gate chamber and the 16 clamshell gates will be by elevator with emergency stairs or ladders. The design includes furnishing and installing one geared electric traction freight elevator with a capacity of 3,500 lbs. The elevator will have two landings with a car size of 8'-0" by 8'-0" and a travel of approximately 60 feet.

Access to the hoist equipment will be by stairs and walkways located over the stilling basin and supported by the piers.

Electrical

The electrical features and equipment for this alternative include a centralized control board, a power distribution panel board, gallery lighting, and all conduit, cable, and grounding to complete the installation.

The control board would probably be located at some convenient location within the Third Powerplant. A selector switch would be provided to allow operation locally at each gate or valve, at the centralized control board, or remotely at the main control room. A set of OPEN and CLOSE push buttons would be provided for each of the 16 clamshell gates to operate any gate from the control board. The motor operator at each gate would also contain push buttons for local operation at that particular gate. The wheel mounted gate would be operated locally by the existing gantry crane.

Power requirements for the gates, hoists, air supply, and lighting are estimated to be between 100-150 kVA at a supply voltage of 480 volts. It is assumed that the station service system within the Third Powerplant could accommodate a feeder of this size to service this power load. A 480 volt distribution panel(s) would provide power to the various gates, hoists, air supply, and lighting system. The conduit, cabling, and grounding systems needed to complete the electrical system has been evaluated in this feasibility study and does not present any significant design problems. These systems would be fully designed during the final design process.

Diversion Requirements

The construction of two cofferdams will be required. One cofferdam will be required in the forebay and one in the tailrace adjacent to the Third Powerplant. The forebay cofferdam is anticipated to be a cellular cofferdam, 180 feet high and 220 feet long. Construction and use of this cofferdam will block a minimum of 1 unit of the Third Powerplant. Power generation at the Third Powerplant will be reduced by a minimum of one unit while the cofferdam is in use and construction of the intake for the forebay cascade tunnel is ongoing.

The second cofferdam will be constructed in the tailrace adjacent to the Third Powerplant. The cofferdam will not be as tall, but will be longer than the tailrace cofferdam of the conceptual designs. This cofferdam will be approximately 40 feet high and 1520 feet long. The purpose of this cofferdam will be to allow construction of the gate chamber, stilling well basin, baffled cascade, and adjustable stilling basin with piers. This cofferdam is presently designed as a cellular cofferdam to limit the projected footprint of the structure and to limit the impact on the power production at the Third Powerplant.

Hydraulic and Total Dissolved Gas Evaluation

Hydraulic Analysis

The hydraulic model study and analysis for this alternative consisted of determining acceptable tunnel size, manifold chamber design, gate design, stilling well geometry, baffled cascade geometry to strip supersaturated gas or degas the water, and stilling basin design to prevent replunging of the flow at the toe of the structure.

A 40-ft-diameter tunnel was designed to be equal in size to the existing penstocks in the Third Powerplant. The maximum velocity is 40 ft/s in the tunnel while passing the design discharge. The wheel mounted gate will be used only fully open or closed and will not provide control. An air valve is needed downstream from the wheel-mounted gate to allow air to evacuate or fill from the tunnel when filling or emptying, respectively.

Early in the model design process, it became apparent that the flow velocities produced by the high head were too large for effective manifold and gate design. Prior to model construction, the concept slide gate and butterfly valve, emergency and control gates, respectively, were replaced by controlling clamshell gates at the pipe exits. The clamshell gates are better suited for controlling the high-velocity submerged flow from the manifold.

The manifold acts as a chamber for the exit pipes distributing the flow from the 40-ft-diameter tunnel. The manifold design was investigated using the pipe network program “Fathom”[12]. Reservoir Els. 1290 and 1208 were used with head loss computations to the tunnel upstream from the manifold as input to the program. The initial resulting output was unable to produce a design that prevented significant subatmospheric pressures in each of the exit pipes downstream from the junction with the manifold chamber. Therefore, it became apparent that the 40-ft diameter pipe section would need to dissipate energy prior to the flow entering the manifold. Additional head loss was attained by designing four 35.75-ft-orifice rings on 225 ft centers along the length of the 40-ft-diameter tunnel from the forebay at centerline El. 1130 to upstream from the manifold at centerline El. 980. Head losses were then recomputed through the trashrack, gate, orifice rings and tunnel to the point of measurement upstream from the manifold on the tunnel. The differential head ranges from Els. 1208 to 980 and El.1290 to 980 were reduced from 228 and 310 feet to 104 and 186 feet, respectively, making the manifold and pipe exit design feasible. The minimum head differential with clamshell gate coefficients and the design discharge of 50,000 ft^3/s was used to determine the number and size of exit pipes required. The maximum head differential was used to investigate velocities and pressures at the exit pipe junction with the manifold. To meet discharge capacity requirements and prevent a significant pressure drop at the entrance of each exit pipe, the pipes were designed to have a larger opening at the junction with the manifold and a conical shape with 12 feet to 7.5 feet reducing sections from the manifold to the gates. Twenty five exit pipes on 30 foot centers were designed with the most upstream pipe located 50 feet downstream from the beginning of the manifold. The program predicted flow out of each pipe of about 2000 ft^3/s at reservoir El. 1208. The clamshell gates needed to be throttled to about 60 percent open to restrict the flow at reservoir El. 1290.

This manifold, exit pipe geometry and gates modeled as slide gates were included in the model for initial testing, figure 41. The reservoir was modeled with a head tank. Pressure head measurements were made in the pipeline upstream of the manifold chamber to accurately represent the reservoir head for the given discharge. Table 6, of clamshell gate coefficients, was used to compute a discharge rating curve for the clamshell gates.

Table 6. Discharge coefficients for percent gate opening for a clamshell gate.

Percent Open	C_d	$K = 1/(C_d)^2$
10	0.08	156.25
20	0.17	34.6
30	0.26	14.79
40	0.35	8.16
50	0.45	4.94
60	0.56	3.19
70	0.66	2.3
80	0.77	1.69
90	0.88	1.29
100	1.00	1.00

Initial testing in the model revealed that the flows from the fully open gates at reservoir El. 1208 were significantly greater than estimated by the computer program. In addition, the manifold pressures were very constant throughout the structure and the pressures at the exit pipe were all positive relieving concerns about potential cavitation [8]. The difference between the model and program flow prediction was investigated. After reviewing the program computations, it was determined that too much head loss was used at the exit of each pipe, producing conservative discharge values. As a consequence, several exit pipes and gates were not used to pass flow. Only 16 fully open gates were required to pass the design value at reservoir El. 1208. The location of the open gates along the length of the manifold was tested in the model to balance the flow conditions and achieve uniform flow conditions in the stilling well. In the prototype the exit pipe spacing would be adjusted within the total width of the well maintaining the full width.

Stilling well modifications were made during the study to reduce the turbulence and boiling from the high velocity jets issuing from the clamshell gates. Modifications included increasing the depth of the well and adding a fillet to redirect the high-velocity flow from the gates downward. The elevation of the stilling well was decreased from 960, in the conceptual design, to El. 926 or an increase in depth of 34 ft. In addition, a continuous fillet was installed at El. 983.7 throughout the stilling well below the crest on the wall opposite from the gates and on the end wall. This fillet redirects the flow from the gates downward making use of the stilling well depth for energy dissipation. In addition, the fillet assists with making the surface less turbulent and spreads the flow more uniformly within the well prior to flow over the broad crest weir. More pressure head and turbulence at the downstream end of the stilling well due to the manifold effect required adding the fillet at the downstream end of the stilling well. In addition, the length of the manifold and cascade should be increased by 20 ft to assist with flow conditions at the downstream end of the stilling well and over the left side of the cascade. The diffuser covering the stilling well proposed in the concept design was not needed after these modifications were made.

The discharge from each gate is approximately 3125 ft^3/s and the accompanying velocity about 70 ft/s into the well. The jet from the 7.5 foot diameter gate will impact the far wall of the stilling wall, 50 feet away. Using equation 3, the core of the jet will have decayed somewhat but will still impact on the far wall. The wall will be formed of reinforced concrete over mass concrete, therefore, stability should not be an issue. No additional protection has been provided for abrasion or cavitation damage as it is felt to not be a concern.

The elevation of the crest of the cascade was chosen during the concept design to provide adequate drop for stripping gas before entering the tailwater. The design head, H, over the cascade was, at the same time, limited to 10 feet to ensure reasonable flow depths and unit discharges for effective degassing. For the concept design, the total width of the cascade that would fit in the north service yard was chosen as 800 feet. Using a discharge coefficient of 2.6 for a broad crested weir the head of 8.4 feet was computed using:

$$Q = CLH^{1.5} \qquad (5)$$

where Q= 50,000 ft^3/s; L=800 feet; C = discharge coefficient.

The resulting unit discharge was 62.4 ft^3/s for the 800 foot cascade. The model was constructed to this width, but the recommended width is 820 feet after stilling well modifications. An 820 foot width produces a unit discharge is 61 ft^3/s per linear foot for the design discharge of 50,000 ft^3/s.

The stepped cascade did not adequately dissipate energy over the 40 foot drop from the crest at El. 1012 to the toe of the 2:1 slope at El. 972. In addition, the inception point for the turbulent boundary layer to reach the surface, thus producing gas stripping would not occur until about half the distance down the slope at 45.5 feet [13]. Several alternatives were investigated, including a baffled apron drop and St. Anthony Falls (SAF) stilling basins positioned at various levels. The large tailwater variation did not permit the effective use of fixed dissipators at different elevations within the length allotted for the structure. Therefore, baffled blocks were installed on a section of the 2:1 sloping cascade to increase turbulence and dissipate additional energy before entering the tailwater and/or other dissipator options investigated. Numerous drop and tailwater transitional structures were studied. It was found to be very difficult to prevent plunging of the exiting flow into the tailwater over the anticipated range of tailwater elevations. Figure 42 shows the model operating with the baffled cascade near the wall and the stepped cascade in the remaining section.

The vertically adjusting stilling basin was developed to dissipate energy and prevent plunging of the flow into the tailwater pool over the range of tailwater elevation from 960 to 985 (figure 42). The vertically adjusting basin provides energy dissipation, a transition from the baffled drop in the cascade to the tailwater, gas stripping, and minimizes plunging of the jet off the end of the stilling basin to depth under varying tailwater conditions. The baffle blocks on the stilling basin floor provide additional energy dissipation and gas stripping. The depth of water in the adjustable stilling basin is limited to a maximum of 5 feet to minimize TDG production.

One area of design that was not addressed in the model study was the dynamic loading on the tunnel and encasement during the filling and evacuation phases. Provisions for dewatering the tunnel, manifold, and stilling well for inspection and maintenance would have to be addressed in final design.

Total Dissolved Gas Evaluation

During the conceptual phase it was thought that the stepped cascade design would adequately dissipate energy, reduce TDG levels, and prevent plunging of the flow into the tailwater. Study of the stepped cascade drop and the transitional stepped structure to the tailwater found the energy levels in the flow at the toe of the 2 to 1 and 6 to 1 stepped slopes to be excessive. Sufficient energy was available to form a hydraulic jump where the jet entered the tailwater pool and generated unacceptable plunging and TDG levels.

Several options were identified to further dissipate energy including use of a baffled drop instead of a stepped drop and use of small stilling basins positioned at various elevations on the slope. The length of the cascade structure was also increased. Many options were approximated in the model and evaluated to determine their TDG characteristics. TDG levels could not be successfully controlled with fixed structure concepts under the highly varying tailwater levels.

The TDG evaluation focused on the stilling basin, baffled cascade, and tailwater immediately downstream from the stilling basin apron. Recirculation of the flow and minimal surface turbulence in the stilling well may very well increase TDG levels, however, the baffled portion of the cascade provided excellent gas stripping. Flow over the baffled 2:1 sloping cascade was highly turbulent and shallow producing both effective energy dissipation and gas stripping, figure 42. Flow over the stilling basin floor would be constant at 5 ft, regardless of the tailwater elevation. The apron length of 50-ft downstream from the end of the baffled section was adequate to establish the direction of the flow exiting the apron.

The stilling basin floor, in concept, acts as a raised tailrace [14]. It supplies a shallow transitional structure in which free air bubbles rise and clear from the flow. Additional degassing or a least TDG stabilization could occur in this zone. The COE has investigated the raised and shallow tailrace concept at Lower Granite, Ice Harbor, and The Dalles Dams and their influence on TDG. Although COE findings are not fully applicable for design and prediction of gas transfer characteristics of the basin, the findings show the shallow runout to be a beneficial feature. Unfortunately, some replunging of the flow still occurred at the end of the stilling basin apron.

The TDG characteristics of the design were determined by visual observations of the flow conditions over the cascade stilling basin and into the tailwater. The tailwater was varied by 27 feet from El. 958 to El. 985 which would be generated by a 50,000 ft^3/s spill with 30,000 ft^3/s to 280,000 ft^3/s powerplant release, respectively. The 7Q10 total release from the dam of 241,000 ft^3/s yields a corresponding tailwater of El. 975.

Flow conditions were documented leaving the stilling basin apron for the design flow of 50,000 ft^3/s under reservoir Els. 1208 and 1290 with tailwater elevations ranging from 965 to 985 feet. The available energy was less at reservoir El. 1208 than at 1290, however, the stilling well dissipated energy satisfactorily for both cases, producing a constant available head over the cascade. The lower tailwater elevations produced the greatest drop, and thus yielded the highest velocities and most potential for replunging into the tailwater. The depth over the apron will always be maintained at 5 ft regardless of tailwater elevation. Dye tracings at the end of the apron and in the tailwater pool are shown in figures 43 through 50 for reservoir Els. 1208 and 1290 and tailwater Els. 965 and 985. Figures 43 through 46 show dye injections for reservoir El. 1208. Figures 47 through 50 show dye injections for reservoir El. 1290. Dye injections were made at both the surface and the floor at the end of the apron. The 5-ft depth of flow over the apron will produce free venting of the air bubbles and reduce TDG levels to about 110 percent. Unfortunately, as may be seen by the dye tracings in figures 43 through

50, vertical mixing to shallow depths appears to occur off the end of the apron. A conservative approach was taken to evaluating the TDG characteristics by assuming that air bubbles would be drawn to the depth shown by the dye.

To assist in the TDG evaluation, average velocities were measured downstream from the end of the apron for reservoir El. 1290, figure 51. The average velocities were 8.9 and 7.4 ft/s for tailwater Els. 965 and 985, respectively. These velocities were apparently high enough to produce vertical mixing to a depth of about 12 feet confirming the dye tracings. Therefore, the expected TDG levels produced by spill over the forebay pipe with cascade alternative are expected to be 125 percent with a total discharge of 50,000 ft^3/s regardless of reservoir head or tailwater elevation. This TDG level from the spill would be mixed with powerplant releases to produce total river TDG levels as shown in Table 7 and figure 52.

Two approaches can be used to further reduce the unit discharge over the cascade and thus improve TDG reduction characteristics. These include:

- widening the cascade
- reducing the design discharge keeping the width the same

The cascade may only be widened an additional 180 feet in the area of the north service yard. This would reduce the unit discharge to 50 $ft^3/s/ft$. Model investigations of vertical mixing in the tailwater at the end of the apron showed only a minimal TDG benefit at this unit discharge. Therefore, the model was operated at progressively smaller discharges until optimal performance was obtained. This occurred at a flow rate of 20,000 ft^3/s or a unit discharge of 24.4 $ft^3/s/ft$ where the TDG levels would reach 110 percent for the entire range of tailwater conditions.

The cascade width required to produce the optimal TDG level of 110 percent is 2000 feet. This is not physically practical. The width of this alternative should be as large as possible recognizing that performance will improve with lower discharges. Reduction of the design spill, possibly by joint operation, would be the most practical way to improve TDG characteristics of this alternative. A river TDG of 110 percent saturation may only be obtained with this alternative when the reservoir TDG levels are 105 percent or less and relatively large flows occur.

Table 7. - TDG levels for the forebay pipe with cascade alternative at reservoir elevation 1208 and 1260 with various lake TDG levels.

Tailwater Elevations	Qs	Qpp	Qt	TDG of OW only	River TDG for various reservoir TDG levels						
					TDG Reservoir						
					100%	105%	110%	115%	120%	125%	130%
	cfs	cfs	cfs	%							
Min. tw El. 958	50000	25000	75000	125	116.67	118.33	120.00	121.67	123.33	125.00	126.67
Normal tw range El. 962-966	50000	72000	122000	125	110.25	113.20	116.15	119.10	122.05	125.00	127.95
Max. tw range El. 966-970	50000	118000	168000	125	107.44	110.95	114.46	117.98	121.49	125.00	128.51
Tw El. 970-972	50000	160000	210000	125	105.95	109.76	113.57	117.38	121.19	125.00	128.81
7Q10 tw El. 975	50000	191000	241000	125	105.19	109.15	113.11	117.07	121.04	125.00	128.96
Tw El. 980	50000	245000	295000	125	104.24	108.39	112.54	116.69	120.85	125.00	129.15
Tw El. 985	50000	280000	330000	125	103.79	108.03	112.27	116.52	120.76	125.00	129.24

Shaded values are below 110 percent TDG supersaturation in the river below Grand Coulee.

Construction Features and Cost Estimate

The forebay pipe with cascade alternative, baffled apron, and adjustable stilling basin will take about 4 years to construct. This alternative will be difficult to construct and operate. There are 48, 5-foot diameter piers that need to be embedded 5 feet into the foundation. The installation of these piers would require large cranes or barges, but could be performed in a wet or dry environment. The construction of the adjustable stilling basin floor will be more involved with construction involving a sequence of steps. The stilling basin floor could be assembled in a contractor use area adjacent to the workday prior to installing them on the piers. The mechanical design and testing of the air supply and hoist system are complex and potentially difficult.

One concern is the location of the contractor's use area, because the north service yard will be in the middle of the construction area. The location for the concrete batch plant and delivery system will require additional study. Disposal of excavated material will also be an issue because there will be approximately 700,000 cubic yards of waste. This could cover an area of 30 acres to depths of 20 feet.

The actual construction efforts for the manifold and the baffled cascade are reasonably conventional using reinforced concrete placement practices. The construction of a 200-ft-tall cofferdam in the reservoir forebay blocking portions of the Third Powerplant will require detailed designs. The cofferdams were estimated as cellular to minimize space required. The work will not permanently impact power production, but the loss of revenue during the construction and use of the forebay cofferdam will be significant. Third Powerplant revenues will be lost during construction of the cofferdam, completion of the new end dam, installation of the fixed wheel gate and trashrack, and removal of the cofferdam. Every effort should be made to complete this portion of the work as quickly as possible to minimize loss of power revenues.

The power loss revenues were computed based on information from the USBR powerplant data contained on the world wide web (www.usbr.gov/power/data). The 10-year average powerplant production at Grand Coulee dam is approximately 22 billion kWh/year and the 1999 production was just under 24 billion kWh. The 1996 historical average priority firm rate is 2.39 cents per kWh. The unit that will be out of service due to the cellular cofferdam in the forebay is unit No. 24 and maybe unit No. 23., which contribute approximately 12 percent each of the total powerplant capability if all units are operating. It was assumed that 12 percent of the average 22 billion kWh times the firm rate is the revenue from unit No. 24 over 1 year. Therefore, the loss of power during construction is the percentage of the year the unit is out of service times the powerplant production of Unit No. 24 for 1 year. Using this analysis, the powerplant revenue loss is estimated at $47,000,000 for one unit. The cost would double if two units were removed from service for the construction of the cofferdam. At this level of estimate, no loss of power revenue is anticipated due to the cofferdam in the tailrace for construction of the cascade, but this work could potentially block a portion of the tailrace.

Unwatering and dewatering design and capital costs for operation and maintenance purposes would be covered in the final design. The cost of these items is covered by the contingencies at this time.

The details of the listed items and costs are shown in appendix 3. The field cost for this alternative using the design flow of 50,000 ft^3/s is estimated at $300,000,000. There are significant differences between this design and the conceptual design including the number and design of the gates, the design of the baffled cascade, and the adjustable stilling basin. The non-contract cost at 30 percent is $90,000,000. Including power revenue losses, the total cost would be $437,000,000. The extent of this alternative would not be modified for smaller or larger discharges, only the unit discharge would change benefitting TDG production for smaller discharges.

Comparisons

Comparisons are made for the design spill of 50,000 ft^3/s for the existing outlet works geometry and spill pattern and all three alternatives that have been under feasibility design investigation. Table 8 shows the computed spill generated and combined river TDG levels for the existing outlet operation and all three alternatives. Comparisons of the effectiveness of the alternatives for TDG can be made by computing the reduction of TDG for each alternative relative to the base TDG levels produced by using the outlet works. Table 8 shows a summary of the data contained in Tables 1, 2, 4, and 7. The difference in TDG percent is a straight comparison to the existing outlet flow condition for each respective reservoir TDG level.

The existing outlet works only will generate TDG levels in the spill up to 190 percent with a spill of 50,000 ft^3/s. The combined TDG in the river with spill from the geometry of the existing outlet works will never be below 110 percent saturation. Each alternative is compared to combined TDG levels in the river for operation of the existing outlet works.

Extending and covering the outlet works, alternative 1, is a transfer alternative. At the 7Q10 design flow event this alternative will generate TDG levels of 119 percent. This alternative will not reduce the TDG to any level below that in the reservoir. There is however, always a reduction in the TDG levels in the river as compared to the existing conditions. The extend and cover alternative will reduce the TDG levels for all flow conditions compared to the existing conditions. The TDG levels in the river decrease with increasing total river flow until the level of the reservoir is matched. The TDG levels will not decrease below that of the reservoir. For the design condition with the 7Q10 flow and approximately 50,000 ft^3/s through the extend and cover alternative, there is an improvement in the TDG of an average of about 14.7 percent until the alternative becomes a transfer alternative with the benefit decreasing to 11.4 percent at a reservoir TDG level of 135 percent.

Constructing deflectors on the downstream face of the spillway, alternative 3, will reduce the TDG from those currently experienced with the outlet works operation. At the 7Q10 design flow event this alternative will generate TDG levels of 128 percent. This alternative can have the greatest variability in TDG reduction effectiveness. This is a function of the varying tailwater and the ability to locate the deflectors where they will be most effective. When the reservoir has a low TDG level, the downstream river will experience decreasing TDG levels with increasing flow. When the TDG level in the reservoir is high, the TDG levels in the river will slightly increase with increasing flow for all conditions. The improvement, hence reduction in TDG for the design flow of 50,000 ft^3/s, averages about 12.9 percent.

The forebay pipe with cascade, alternative 5, is the only alternative that will reduce the TDG in the river below that of the reservoir, when the reservoir is at high TDG levels. At the 7Q10 design flow event this alternative will generate TDG levels of 125 percent. River TDG levels will be reduced when the TDG levels in the reservoir are above 125 percent. This alternative will provide about a 13.5 percent reduction in the TDG levels for the design flows of 50,000 ft^3/s and total river flow of 241,000 ft^3/s.

Table 8. - Expected TDG performance of the existing structure and each alternative for varying initial reservoir TDG levels and the 7-day 10-year flood event.

Alternative	Discharge Values		TDG%		Total Combined TDG%	Difference in TDG%
	Power kcfs	Outlet Spill kcfs	Power	Spill		
Existing Condition	191	50	100	190.00	118.67	0.00
Existing Condition	191	50	105	190.00	122.63	
Existing Condition	191	50	110	190.00	126.60	
Existing Condition	191	50	115	190.00	130.56	
Existing Condition	191	50	120	190.00	134.52	
Existing Condition	191	50	125	190.00	138.49	
Existing Condition	191	50	130	190.00	142.45	
Existing Condition	191	50	135	190.00	146.41	
Extend and Cover	191	50	100	119.00	103.94	-14.73
Extend and Cover	191	50	105	119.00	107.9	-14.73
Extend and Cover	191	50	110	119.00	111.87	-14.73
Extend and Cover	191	50	115	119.00	115.83	-14.73
Extend and Cover	191	50	120	119.00	120.00	-14.52
Extend and Cover	191	50	125	119.00	125.00	-13.49
Extend and Cover	191	50	130	119.00	130.00	-12.45
Extend and Cover	191	50	135	119.00	135.00	-11.41
Deflectors	191	50	100	128.00	105.81	-12.86
Deflectors	191	50	105	128.00	109.77	
Deflectors	191	50	110	128.00	113.73	
Deflectors	191	50	115	128.00	117.70	
Deflectors	191	50	120	128.00	121.66	
Deflectors	191	50	125	128.00	125.62	
Deflectors	191	50	130	128.00	129.59	
Deflectors	191	50	135	128.00	133.55	
Forebay Cascade	191	50	100	125.00	105.19	-13.49
Forebay Cascade	191	50	105	125.00	109.15	
Forebay Cascade	191	50	110	125.00	113.11	

Alternative	Discharge Values		TDG%		Total Combined TDG%	Difference in TDG%
	Power kcfs	Outlet Spill kcfs	Power	Spill		
Forebay Cascade	191	50	115	125.00	117.07	
Forebay Cascade	191	50	120	125.00	121.04	
Forebay Cascade	191	50	125	125.00	125.00	
Forebay Cascade	191	50	130	125.00	128.96	
Forebay Cascade	191	50	135	125.00	132.93	

TDG comparisons to the base existing outlet works flow condition for the three alternatives are shown graphically in figures 53, 54, and 55 for reservoir TDG concentrations of 110, 125, and 130 percent. None of the alternatives reduce TDG in the river to 110 percent or below unless the TDG level in the reservoir is below 110 percent. At a reservoir TDG level of 125 percent, the extend and cover alternative and the forebay cascade alternative will provide the same TDG levels as the reservoir, with the deflector producing a slightly higher level. At a reservoir TDG level of 130 percent, the extend and cover alternative will transfer that value, the deflectors will cause a slight increase, and the forebay cascade a slight decrease. All alternatives provide similar TDG benefit with high reservoir TDG levels with the extend and cover alternative providing a somewhat noticeably lower TDG level under low reservoir TDG levels.

Summary comparisons of the construction times, field and non-contract costs, and the expected TDG ranking are given in Table 9. These are updated values from the concept design report. The TDG rankings are given for the fixed monitoring station located six miles downstream from the dam and are for fully mixed flow for the 7-day 10-year event. The mixed flow is the sum of the weighted design spill and powerplant flow divided by the total flow (equation 1).

Table 9. - Comparison of feasibility level structural alternatives using a design flow of 50,000 ft^3/s for TDG abatement for Grand Coulee Dam.

Alternative	**Construction Duration (years)**	**Cost (millions)**		**TDG Ranking**	**Cost Ranking**	**Sum of Rankings**
		Field	**Total**			
Extend and cover outlets (1)	**3**	**74**	**96.2**	**1**	**2**	**3**
Deflectors - (3)	**3**	**12**	**15.6**	**3**	**1**	**4**
Forebay Pipe with Cascade (5)	**4**	**300**	**437***	**2**	**3**	**5**

* These alternatives have an additional estimated power revenue loss of about 47 million dollars spanning a 9 month construction period.

The best overall modification would be the extend and cover option based upon the lowest combined ranking. The TDG ranking is the best and it is the second least expensive alternative for the design spill of 50,000 ft^{3}/s under the 7-day 10-year event. There are other considerations that may need to also be evaluated before selection of a preferred alternative is made.

Joint Operation of Grand Coulee and Chief Joseph Dams

Joint operation of Grand Coulee (GC) and Chief Joseph (CJ) Dams, owned by Reclamation and the COE, respectively, was called for in the National Marine Fisheries Service 1998 Biological Opinion. The intent of the joint operation is to achieve the most cost-effective way to abate gas between the two closely linked projects. The COE has studied several alternatives. Their recommended plan is to install deflectors on the spillway bays at Chief Joseph Dam and shift power and spill between Grand Coulee and Chief Joseph Dams. The data and plan is outlined in the COE Chief Joseph Gas Abatement Study and General Reevaluation Report (COE GRR) dated April 2000 [4].

The structural alternatives for gas abatement at Grand Coulee Dam have been discussed in the previous sections. No alternative will consistently meet water quality standards of 110 percent for the design value of 50,000 ft^3/s and the total 7-day, 10-year (7Q10) event of 241,000 ft^3/s. Therefore, the concept of minimizing spill by adding power at Grand Coulee is potentially valuable. This concept will be studied with the existing outlet works operation and each of the structural alternatives. To investigate the possibility of spilling less than the design value, each of the outlet works alternatives has been priced per block or pair of outlet works modified with their corresponding discharge capability. The forebay cascade alternative will remain the same width, but the unit discharge may be modified for comparison of different flow rates.

The joint operation scenario requires that the total flow from each project would remain the same, but the relative amounts of power generation and spill would change between the two projects. To maintain the same power production the difference in hydraulic heads and turbine characteristics are accounted for in the shift of power and spill. The hydraulic head at Grand Coulee is about twice that at Chief Joseph. Therefore, when shifting power to Grand Coulee, twice as much power can be produced for the same amount of flow and the reduction in spill at Grand Coulee would amount to one-half the total flow volume transferred at Chief Joseph. Reducing the TDG below Grand Coulee by releasing more flow through the power plant will have the effect of lowering the TDG levels below Grand Coulee and also lowering the initial TDG levels at Chief Joseph. Even though there is more flow released through the spillways at Chief Joseph, the total increase in TDG is lower because of the deflectors and the lower initial TDG levels.

Two approaches were used to determine the benefit of transferring power to Grand Coulee and spill to Chief Joseph for the existing outlet works flow conditions and each of the alternatives:

- use the COE GRR report to evaluate the effect on the TDG below Grand Coulee for joint operation
- select various flow rates for transfer and evaluate the TDG benefit obtained by a straight reduction in spill

Flow Transfer Evaluation

The first approach was to use the COE GRR report that gives a comprehensive analysis of the spill transfer between the projects out of the 7Q10 total based on the hourly data from the 1997 flow event [4]. They summarized the results into a single average value of TDG below Grand Coulee for the existing outlet works operation of about 123 percent and about 115 percent with spill transfer for the 1997 flow event. This would be an improvement of about 8 percent on average.

To evaluate more in depth the existing outlet operation and that of each structural alternative information in the COE GRR was used to determine the amount of flow that can be transferred between the projects out of the 7Q10 total based on the capacities of both power plants, the power demand and the hourly data from the 1997 flow event. This first approach is perhaps more comprehensive displaying influence achieved for one specific spill season. Spill flow exceedences could certainly be quite different for a lesser water year.

Figure 56 shows the data used in the analysis to determine the flow transfer. The 1997 observed data, the operational spill data and the total spill curves are for Chief Joseph flow and were replicated from plate 3-7a of the COE GRR. Grand Coulee power plant capacity was assumed to be 250,000 ft^3/s and Chief Joseph power capacity was assumed to be 200,000 ft^3/s, both less than total capacity. The percent exceedence of spill reflects only the flow data at or below the 7Q10 value of 241,000 ft^3/s experienced at Chief Joseph Dam for the spill season of March 1 through June 30, 1997. About 75 percent of the total data for the 1997 spill season fell below the 7Q10 value of 241,000 ft^3/s, therefore it was felt appropriate to use this data in the analysis. This reflects the position that TDG standards are not enforced for discharges greater than the 7Q10 event. The difference between the 1997 observed spill and the total spill is the flow through the power plant at Chief Joseph. For joint operation, power is transferred to Grand Coulee to increase power generation and reduce spill. The operational spill at Chief Joseph is increased above that observed to account for lesser power production at Chief Joseph and flow transfer. Flow that would normally be used to produce power at Chief Joseph would be transferred to Grand Coulee and the differing hydraulic heads allow about half the flow from Chief Joseph to produce the same power at Grand Coulee, thus requiring spill to increase at Chief Joseph for the remaining flow. These curves were all developed by the COE.

For the Grand Coulee analysis, it was assumed that the difference between the observed and operational spill curves in figure 56 would be the power shift that would be transferred to Grand Coulee Dam as a function of the percent exceedence of spill. The second y-axis on figure 56 was used to plot the flow representing the difference between the operational and observed spills at Chief Joseph and the flow transferred to Grand Coulee. The total flow transferred, shown as increase CJ spills, must then be separated into power generation and spill at Grand Coulee Dam. Twice as much power is produced as Grand Coulee for the same flow and power production between the two projects must remain the same. Therefore the total amount of flow that reduces spill at Grand Coulee is one half the flow transferred or the ½ CJ spill curve.

Figure 57 shows the Grand Coulee spill, GCD spill flows, obtained by subtracting ½ CJ spill flows from figure 56 from the design flow of 50,000 ft^3/s to obtain a percent of a spill flow will be exceeded. The power plant flow curve, GCD PP flow, is the difference between the 50,000 ft^3/s design discharge and the reduced spill volume due to a joint operation. This GCD

PP flow would be added to the existing power produced at Grand Coulee for the 7Q10 event up to the chosen cap in the analysis of 250,000 ft^3/s.
Therefore, the GCD spill curve data on figure 57 represents the spill transfer values, referenced to the 7Q10 event, that were then used to evaluate the effect of joint operation on the TDG in the river below Grand Coulee for the existing outlet works operation and each of the proposed structural modifications.

With the transfer spill defined as a percent exceedence of spill, the analysis could be performed on the existing outlet works and each of the structural alternatives. Data shown in Table 10 include the percent of time each spill transfer flow would be experienced, the various flow amounts, and the effect on the TDG in the river for the existing outlet works flow condition with joint operation. The first column is the percent of time the spill flow would be exceeded. The next four columns are the various flows used in the computation, including the design flow, the reduced spill flow volume, the increased flows through the power plants, and the total power discharge at Grand Coulee. TDG levels for the existing outlet works flow condition with flow transfer are shown graphically on figure 58. The effect of various amounts of spill on reducing the TDG for the existing outlet works may be evaluated using figure 58 and compared to the average values previously reported. The left y-axis shows the design flow and the proportioned spill and power from figure 57. On the right vertical axis is the percentage of TDG for mixed flow conditions in the river below Grand Coulee Dam assuming initial TDG levels in the reservoir varying from 100 to 135 percent in 5 percent increments. This graph allows one to evaluate the TDG benefit as a percent of time that the particular flow rate selected is exceeded. These TDG and spill values can then be compared to those of the existing outlet works operation under the design flow rate to determine the benefit for a wide range of operational scenarios.

The resulting TDG levels computed for the reduced spill for the extend and cover alternative are shown in Table 11 and graphically in figure 59. Data shown in Table 11 include the weighted flow volumes, the reduction in the number of outlet pairs or blocks modified as transfer increases, and the effect on the TDG in the river for the extend and cover alternative with joint operation. Without flow transfer, the TDG production level of the outlet works extend and cover alternative is 119 percent for the 7Q10 value and eight outlet pairs would need to be modified at a cost of $96.2 million. Combined river TDG would be 120 percent for a 120 percent lake TDG level. Along the GCD spill flow curve are the number of outlet pairs that would need to be modified to pass the flow rate shown. If it was decided to allow a spill flow exceedence of 25 percent of the time, then the spill flow would be 33,000 ft^3/s, the total power release would be 208,000 ft^3/s, a reservoir TDG level of 110 percent, the mixed TDG levels in the river would be 111 percent (Table 11 and figure 59). Redesigning and using a lower design spill flow of 33,000 ft^3/s would mean that six pairs of outlets would be modified for a cost of $82 million, thus saving $14.2 million (see Table 3). For this option, once the TDG level of the reservoir exceeds that of the spill TDG production, then the combined TDG level is constant and equal to the reservoir TDG level. This type of comparison can be achieved for whatever level of flow is chosen as an acceptable design spill flow. The indication is that even with a full transfer of 50,000 ft^3/s there is no overall effect on the TDG below Grand Coulee Dam unless the initial level of TDG in the reservoir is below 105 percent.

TDG levels for flow transfer for the deflector alternative are shown in Table 12 and figure 60. The deflectors alone will produce a TDG level of 128 percent with six outlet pairs modified for the design flow of 50,000 ft^3/s under the 7Q10 event. Mixed river TDG without transfer would be 122 percent using an initial 120 percent lake TDG level. Joint operation would allow construction of less deflectors at Grand Coulee as shown on figure 60 along the GCD spill curve. For example, at a spill flow of about 14,000 ft^3/s that would be exceeded 55 percent of the time, only two blocks would need to be modified. If the reservoir had a TDG level of 100 percent then the combined river TDG level would be about 102 percent. If the reservoir had a TDG level of 120 percent, then the combined river TDG level would be about 120 percent. Modifying only 2 outlet pairs would cost $6.5 million, saving $9.6 million over modifying six outlet pairs (Table 5). The TDG levels are reduced for the 7Q10 event with the deflector alternative only if the reservoir TDG is below 110 percent and also because the deflector produces fairly high TDG levels compared to the reservoir levels. This graph can be used to determine the acceptable level of TDG in the river for a given risk or frequency of spill for the deflector alternative. However, the overall improvement under the joint operation using the 1997 data and percent of exceedence of spill is relatively minor regardless of the amount of spill.

The TDG computations for flow transfer for the forebay pipe with cascade alternative are shown in Table 13 with graphical results on figure 61. Unlike the extend and cover alternatives, the size of the forebay cascade would remain the same as the discharge decreased. The unit discharge would be decreased allowing a decrease in the TDG production for the same size structure. The same TDG characteristics could be maintained and the length of the cascade shortened, however, this would not be overly beneficial. Thus, this alternative does not become more cost effective, but can reduce gas production with the expected reduced spill. It should be noted that the relative improvement is most significant with small spills and when the reservoir TDG level is below 110 percent. The most improvement is for large transfer amount or very small spills. An acceptable design spill flow may be selected and the TDG in the river determined for the frequency of event chosen.

TDG as a Function of Fixed Flow Transfer

This previous approach is somewhat cumbersome and limited by the use of only one year of data in its development. Therefore, a second approach was investigated. The second approach was to simply compute the TDG levels and reduced structural modifications, thus cost savings, from a given spill reduction at Grand Coulee of 10,000 and 20,000 ft^3/s. The second approach is perhaps oversimplified but allows a more direct comparison to TDG reduction benefits as predicted by the approach previously presented. Flow rates of 10,000 and 20,000 ft^3/s were chosen as reasonable amounts based upon the spill curve from figure 57. A 10,000 ft^3/s transfer would apply for 75 percent of the time, and a 20,000 ft^3/s transfer would apply approximately 37 percent of the time.

Table 10. - TDG analysis of the existing outlet works flow condition with joint operation for the 7Q10 event of 241,000 ft^3/s.

TDGspill = 190%

percent of time	Design Q cfs	GCD spill flows	GCD Qpp added flow	GCD spill cfs	River TDG for various reservoir TDG levels — Reservoir TDG 100	105	110	115	120	125	130	135
0	50000	50000	0	191000	118.67	122.63	126.60	130.56	134.52	138.49	142.45	146.41
5	50000	40000	10000	201000	114.94	119.11	123.28	127.45	131.62	135.79	139.96	144.13
10	50000	38000	12000	203000	114.19	118.40	122.61	126.83	131.04	135.25	139.46	143.67
15	50000	35750	14250	205250	113.35	117.61	121.87	126.13	130.38	134.64	138.90	143.16
20	50000	35000	15000	206000	113.07	117.34	121.62	125.89	130.17	134.44	138.71	142.99
25	50000	33000	17000	208000	112.32	116.64	120.95	125.27	129.59	133.90	138.22	142.53
30	50000	29250	20750	211750	110.92	115.32	119.71	124.10	128.50	132.89	137.28	141.68
35	50000	22000	28000	219000	108.22	112.76	117.30	121.85	126.39	130.93	135.48	140.02
40	50000	17750	32250	223250	106.63	111.26	115.89	120.52	125.16	129.79	134.42	139.05
45	50000	15750	34250	225250	105.88	110.55	115.23	119.90	124.57	129.25	133.92	138.59
50	50000	14500	35500	226500	105.41	110.11	114.81	119.51	124.21	128.91	133.61	138.31
55	50000	14000	36000	227000	105.23	109.94	114.65	119.36	124.07	128.78	133.49	138.20
60	50000	12250	37750	228750	104.57	109.32	114.07	118.81	123.56	128.30	133.05	137.80
65	50000	10500	39500	230500	103.92	108.70	113.49	118.27	123.05	127.83	132.61	137.40
70	50000	10500	39500	230500	103.92	108.70	113.49	118.27	123.05	127.83	132.61	137.40
75	50000	10000	40000	231000	103.73	108.53	113.32	118.11	122.90	127.70	132.49	137.28
80	50000	9750	40250	231250	103.64	108.44	113.24	118.03	122.83	127.63	132.43	137.23
85	50000	8750	41250	232250	103.27	108.09	112.90	117.72	122.54	127.36	132.18	137.00
90	50000	7500	42500	233500	102.80	107.65	112.49	117.33	122.18	127.02	131.87	136.71
95	50000	5000	45000	236000	101.87	106.76	111.66	116.56	121.45	126.35	131.24	136.14
100	50000	1250	48750	239750	100.47	105.44	110.41	115.39	120.36	125.34	130.31	135.29

These data are represented graphically on figure 58.

Table 11. - TDG analysis of the extend and cover alternative with joint operation for the 7Q10 event of 241,000 ft^{3}/s.

Total River Q 241000 cfs

TDG at El. 1260 spill 241kcfs, tw= 975 = 119 %

						TDG reservoir							
						100	105	110	115	120	125	130	135
percent of time	Design Q spill	GCD flows	GCD Qpp	Plus Qpp	No. of blocks	River TDG for various reservoir TDG levels							
0	50000	50000	0	191000	8	103.94	107.90	111.87	115.83	120.00	125.00	130.00	135.00
5	50000	40000	10000	201000	7	103.15	107.32	111.49	115.66	120.00	125.00	130.00	135.00
10	50000	38000	12000	203000	7	103.00	107.21	111.42	115.63	120.00	125.00	130.00	135.00
15	50000	35750	14250	205250	6	102.82	107.08	111.34	115.59	120.00	125.00	130.00	135.00
20	50000	35000	15000	206000	6	102.76	107.03	111.31	115.58	120.00	125.00	130.00	135.00
25	50000	33000	17000	208000	6	102.60	106.92	111.23	115.55	120.00	125.00	130.00	135.00
30	50000	29250	20750	211750	5	102.31	106.70	111.09	115.49	120.00	125.00	130.00	135.00
35	50000	22000	28000	219000	4	101.73	106.28	110.82	115.37	120.00	125.00	130.00	135.00
40	50000	17750	32250	223250	3	101.40	106.03	110.66	115.29	120.00	125.00	130.00	135.00
45	50000	15750	34250	225250	3	101.24	105.91	110.59	115.26	120.00	125.00	130.00	135.00
50	50000	14500	35500	226500	3	101.14	105.84	110.54	115.24	120.00	125.00	130.00	135.00
55	50000	14000	36000	227000	3	101.10	105.81	110.52	115.23	120.00	125.00	130.00	135.00
60	50000	12250	37750	228750	2	100.97	105.71	110.46	115.20	120.00	125.00	130.00	135.00
65	50000	10500	39500	230500	2	100.83	105.61	110.39	115.17	120.00	125.00	130.00	135.00
70	50000	10500	39500	230500	2	100.83	105.61	110.39	115.17	120.00	125.00	130.00	135.00
75	50000	10000	40000	231000	2	100.79	105.58	110.37	115.17	120.00	125.00	130.00	135.00
80	50000	9750	40250	231250	2	100.77	105.57	110.36	115.16	120.00	125.00	130.00	135.00
85	50000	8750	41250	232250	2	100.69	105.51	110.33	115.15	120.00	125.00	130.00	135.00
90	50000	7500	42500	233500	2	100.59	105.44	110.28	115.12	120.00	125.00	130.00	135.00
95	50000	5000	45000	236000	1	100.39	105.29	110.19	115.08	120.00	125.00	130.00	135.00
100	50000	1250	48750	239750	1	100.10	105.07	110.05	115.02	120.00	125.00	130.00	135.00

These data are represented graphically in figure 59.

Table 12. - TDG Analysis of the deflector modification with joint operation with the 7Q10 event.

Total River Q = 241000 cfs
TDG=128%, Tailwater of 975

percent of time	Design Q cfs	GCD flows	GCD Qpp	no. of blocks	River TDG for various reservoir TDG levels							
				TDG reservoir	100	105	110	115	120	125	130	135
0	50000	50000	0	6	105.81	109.77	113.73	117.70	121.66	125.62	129.59	133.55
5	50000	40000	10000	5	104.65	108.82	112.99	117.16	121.33	125.50	129.67	133.84
10	50000	38000	12000	5	104.41	108.63	112.84	117.05	121.26	125.47	129.68	133.90
15	50000	35750	14250	4	104.15	108.41	112.67	116.93	121.19	125.45	129.70	133.96
20	50000	35000	15000	4	104.07	108.34	112.61	116.89	121.16	125.44	129.71	133.98
25	50000	33000	17000	4	103.83	108.15	112.46	116.78	121.10	125.41	129.73	134.04
30	50000	29250	20750	4	103.40	107.79	112.18	116.58	120.97	125.36	129.76	134.15
35	50000	22000	28000	3	102.56	107.10	111.64	116.19	120.73	125.27	129.82	134.36
40	50000	17750	32250	2	102.06	106.69	111.33	115.96	120.59	125.22	129.85	134.48
45	50000	15750	34250	2	101.83	106.50	111.18	115.85	120.52	125.20	129.87	134.54
50	50000	14500	35500	2	101.68	106.38	111.08	115.78	120.48	125.18	129.88	134.58
55	50000	14000	36000	2	101.63	106.34	111.05	115.76	120.46	125.17	129.88	134.59
60	50000	12250	37750	2	101.42	106.17	110.91	115.66	120.41	125.15	129.90	134.64
65	50000	10500	39500	2	101.22	106.00	110.78	115.57	120.35	125.13	129.91	134.70
70	50000	10500	39500	2	101.22	106.00	110.78	115.57	120.35	125.13	129.91	134.70
75	50000	10000	40000	2	101.16	105.95	110.75	115.54	120.33	125.12	129.92	134.71
80	50000	9750	40250	2	101.13	105.93	110.73	115.53	120.32	125.12	129.92	134.72
85	50000	8750	41250	1	101.02	105.84	110.65	115.47	120.29	125.11	129.93	134.75
90	50000	7500	42500	1	100.87	105.72	110.56	115.40	120.25	125.09	129.94	134.78
95	50000	5000	45000	1	100.58	105.48	110.37	115.27	120.17	125.06	129.96	134.85
100	50000	1250	48750	1	100.15	105.12	110.09	115.07	120.04	125.02	129.99	134.96

These data are presented graphically in figure 61.

Table 13. - TDG analysis for the forebay pipe with cascade alternative with joint operation for the 7Q10 event.

Total River Q = 214,000 cfs

Percent of time	Design Q cfs	GCD flows	GCD Qpp	%TDG spill	River TDG for various reservoir TDG levels							
				TDG reservoir	100	105	110	115	120	125	130	135
0	50000	50000	0	125	105.19	109.15	113.11	117.07	121.04	125.00	128.96	132.93
5	50000	40000	10000	122	103.65	107.82	111.99	116.16	120.33	124.50	128.67	132.84
10	50000	38000	12000	120.8	103.28	107.49	111.70	115.91	120.13	124.34	128.55	132.76
15	50000	35750	14250	119.4	102.88	107.14	111.39	115.65	119.91	124.17	128.43	132.69
20	50000	35000	15000	119	102.76	107.03	111.31	115.58	119.85	124.13	128.40	132.68
25	50000	33000	17000	117.8	102.44	106.75	111.07	115.38	119.70	124.01	128.33	132.64
30	50000	29250	20750	115.7	101.91	106.30	110.69	115.08	119.48	123.87	128.26	132.66
35	50000	22000	28000	112.8	101.17	105.71	110.26	114.80	119.34	123.89	128.43	132.97
40	50000	17750	32250	111.6	100.85	105.49	110.12	114.75	119.38	124.01	128.64	133.28
45	50000	15750	34250	111.2	100.73	105.41	110.08	114.75	119.42	124.10	128.77	133.44
50	50000	14500	35500	110.9	100.66	105.35	110.05	114.75	119.45	124.15	128.85	133.55
55	50000	14000	36000	110.8	100.63	105.34	110.05	114.76	119.47	124.18	128.88	133.59
60	50000	12250	37750	110.5	100.53	105.28	110.03	114.77	119.52	124.26	129.01	133.75
65	50000	10500	39500	110.2	100.44	105.23	110.01	114.79	119.57	124.36	129.14	133.92
70	50000	10500	39500	110.2	100.44	105.23	110.01	114.79	119.57	124.36	129.14	133.92
75	50000	10000	40000	110	100.41	105.21	110.00	114.79	119.59	124.38	129.17	133.96
80	50000	9750	40250	109.8	100.40	105.19	109.99	114.79	119.59	124.39	129.18	133.98
85	50000	8750	41250	109.7	100.35	105.17	109.99	114.81	119.63	124.44	129.26	134.08
90	50000	7500	42500	109.4	100.29	105.14	109.98	114.83	119.67	124.51	129.36	134.20
95	50000	5000	45000	109.2	100.19	105.09	109.98	114.88	119.78	124.67	129.57	134.46
100	50000	1250	48750	109	100.05	105.02	109.99	114.97	119.94	124.92	129.89	134.87

These data are presented graphically in figure 61.

Tables 14 and 15 and figures 62 and 63 show the results for the existing outlet works flow condition with joint operation. For a spill discharge of 40,000 ft^3/s and 30,000 ft^3/s, TDG levels for the existing outlet flow condition for the 7Q10 event vary from about 115 to 111 percent with reservoir TDG of 100 percent to 144 to 142 percent with reservoir TDG of 135 percent, respectively. Comparatively, these reduced TDG levels could be considered significant between the two flow transfer volumes.

Tables 16 and 17 and figures 64 and 65 show the results for the extend and cover alternative with joint operation. The design flow rate of 50,000 ft^3/s requires eight outlet pairs be modified. By comparison, only seven and five outlet pairs would need to be modified if the spill discharge were reduced to 40,000 and then to 30,000 ft^3/s, respectively. The cost for modifying seven and five outlet pairs would be $88.5 and $72.8 million, respectively. These are $7.7 and $15.4 million less, respectively, compared to the total for the 50,000 ft^3/s design flow cost of $96.2 million. For a spill discharge of 40,000 ft^3/s and 30,000 ft^3/s, TDG levels for the extend and cover alternative for the 7Q10 event vary from about 102 to 103 percent with reservoir TDG of 100 percent to 135 percent with reservoir TDG of 135 percent, respectively. There are only slight differences in the river TDG levels for the joint operation compared to the full design discharge or transfer level.

Tables 18 and 19 and figures 66 and 67 show the results for the deflector alternative with joint operation. The design flow rate of 50,000 ft^3/s requires six outlet pairs or blocks be modified. Reducing the spill discharge to 40,000 and then to 30,000 ft^3/s would require modification of only five and four blocks, respectively. The cost for modifying five and four outlet pairs would be $13.6 and $11.3 million, respectively. These are $2.0 and $4.3 million less, respectively, compared to the total for the 50,000 ft^3/s design flow cost of $15.6 million. For a spill discharge of 40,000 ft^3/s and 30,000 ft^3/s, TDG levels for the deflector alternative for the 7Q10 event vary from about 105 to 103 percent with reservoir TDG of 100 percent to 134 percent with reservoir TDG 135 percent, respectively. There are only slight differences in TDG levels for power transfer with this alternative and this is only a slight improvement over the full 50,000 ft^3/s. TDG levels are reduced by a slightly larger percentage than the extend and cover alternative based upon less spill flow and more power dilution.

Tables 20 and 21 and figures 68 and 69 show the results for the forebay pipe with cascade alternative with joint operation. The forebay pipe with cascade alternative would not reduce in size, hence not reduce in cost, but show benefits in decreased spill TDG production levels from 125 for the full alternative to 122 percent and 116 percent as the unit discharge was decreased to 48.8 $ft^3/s/ft$ for a 40,000 ft^3/s and 36.6 $ft^3/s/ft$ for a 30,000 ft^3/s spill, respectively. There would be no difference in cost for this alternative.

This approach was investigated further with comparisons of TDG benefit due to power transfer for the existing spill condition, the full alternative constructed, and a reduced alternative based on the transfer of power and reducing the effective spill.

Table 14. - TDG analysis for the existing outlet works flow condition with 10,000 ft^3/s transfer from joint operation with Chief Joseph Dam..

Transfer 10kcfs					Reservoir TDG 100	105	110	115	120	125	130	135
	Qs cfs	Qpp cfs	Qt cfs	GCD spill TDG	River TDG with various reservoir TDG levels							
Min. tw El. 958	40000	35000	75000	190	148.00	150.33	152.67	155.00	157.33	159.67	162.00	164.33
Normal tw range El. 962-966	40000	82000	122000	190	129.51	132.87	136.23	139.59	142.95	146.31	149.67	153.03
Max. tw range El. 966-970	40000	128000	168000	190	121.43	125.24	129.05	132.86	136.67	140.48	144.29	148.10
Tw El. 970-972	40000	170000	210000	190	117.14	121.19	125.24	129.29	133.33	137.38	141.43	145.48
7Q10 tw El. 975	40000	201000	241000	190	114.94	119.11	123.28	127.45	131.62	135.79	139.96	144.13
Tw El. 980	40000	255000	295000	190	112.20	116.53	120.85	125.17	129.49	133.81	138.14	142.46
Tw El. 985	40000	290000	330000	190	110.91	115.30	119.70	124.09	128.48	132.88	137.27	141.67

These data are shown graphically on figure 62.

Table 15 . - TDG analysis for the existing outlet works flow condition with 20,000 ft^3/s transfer from joint operation with Chief Joseph Dam..

Transfer 20kcfs					Reservoir TDG 100	105	110	115	120	125	130	135
	Qs cfs	Qpp cfs	Qt cfs	GCD spill TDG	River TDG with various reservoir TDG levels							
Min. tw El. 958	30000	45000	75000	190	136.00	139.00	142.00	145.00	148.00	151.00	154.00	157.00
Normal tw range El. 962-966	30000	92000	122000	190	122.13	125.90	129.67	133.44	137.21	140.98	144.75	148.52
Max. tw range El. 966-970	30000	138000	168000	190	116.07	120.18	124.29	128.39	132.50	136.61	140.71	144.82
Tw El. 970-972	30000	180000	210000	190	112.86	117.14	121.43	125.71	130.00	134.29	138.57	142.86
7Q10 tw El. 975	30000	211000	241000	190	111.20	115.58	119.96	124.34	128.71	133.09	137.47	141.85
Tw El. 980	30000	265000	295000	190	109.15	113.64	118.14	122.63	127.12	131.61	136.10	140.59
Tw El. 985	30000	300000	330000	190	108.18	112.73	117.27	121.82	126.36	130.91	135.45	140.00

These data are shown graphically on figure 63.

Table 16. - TDG analysis for the extend and cover alternative with a 10,000 ft^3/s transfer from joint operation with Chief Joseph Dam.

Transfer 10kcfs					Reservoir TDG							
7 Blocks Modified					100	105	110	115	120	125	130	135
	Qs cfs	Qpp cfs	Qt cfs	GCD spill TDG	River TDG with various reservoir TDG levels							
Min. tw El. 958	40000	35000	75000	131	116.53	118.87	121.20	123.53	125.87	128.20	130.53	135.00
Normal tw range El. 962-966	40000	82000	122000	126	108.52	111.89	115.25	118.61	121.97	125.33	130.00	135.00
Max. tw range El. 966-970	40000	128000	168000	123	105.48	109.29	113.10	116.90	120.71	125.00	130.00	135.00
Tw El. 970-972	40000	170000	210000	122.5	104.29	108.33	112.38	116.43	120.48	125.00	130.00	135.00
7Q10 tw El. 975	40000	201000	241000	119	103.15	107.32	111.49	115.66	120.00	125.00	130.00	135.00
Tw El. 980	40000	255000	295000	115	102.03	106.36	110.68	115.00	120.00	125.00	130.00	135.00
Tw El. 985	40000	290000	330000	112	101.45	105.85	110.24	115.00	120.00	125.00	130.00	135.00

These data are shown graphically in figure 64.

Table 17. - TDG analysis for the extend and cover alternative with a 20,000 ft^3/s transfer from joint operation with Chief Joseph Dam.

Transfer 20kcfs					Reservoir TDG							
5 Blocks Modified					100	105	110	115	120	125	130	135
	Qs cfs	Qpp cfs	Qt cfs	GCD spill TDG	River TDG with various reservoir TDG levels							
Min. tw El. 958	30000	45000	75000	131	112.40	115.40	118.40	121.40	124.40	127.40	130.40	135.00
Normal tw range El. 962-966	30000	92000	122000	126	106.39	110.16	113.93	117.70	121.48	125.25	130.00	135.00
Max. tw range El. 966-970	30000	138000	168000	123	104.11	108.21	112.32	116.43	120.54	125.00	130.00	135.00
Tw El. 970-972	30000	180000	210000	122.5	103.21	107.50	111.79	116.07	120.36	125.00	130.00	135.00
7Q10 tw El. 975	30000	211000	241000	119	102.37	106.74	111.12	115.50	120.00	125.00	130.00	135.00
Tw El. 980	30000	265000	295000	115	101.53	106.02	110.51	115.00	120.00	125.00	130.00	135.00
Tw El. 985	30000	300000	330000	112	101.09	105.64	110.18	115.00	120.00	125.00	130.00	135.00

These data are shown graphically in figure 65.

Table 18. – TDG analysis for the deflector alternative with a 10,000 ft^3/s transfer from joint operation with Chief Joseph Dam

5 blocks modified

Transfer 10kcfs

	Qs cfs	Qpp cfs	Qt cfs	GCD spill TDG	River TDG with various reservoir TDG levels							
Reservoir TDG					100	105	110	115	120	125	130	135
Min. tw El. 958	40000	35000	75000	124	112.80	115.13	117.47	119.80	122.13	124.47	126.80	129.13
Normal tw range El. 962-966	40000	82000	122000	124	107.87	111.23	114.59	117.95	121.31	124.67	128.03	131.39
Max. tw range El. 966-970	40000	128000	168000	125	105.95	109.76	113.57	117.38	121.19	125.00	128.81	132.62
Tw El. 970-972	40000	170000	210000	126	104.95	109.00	113.05	117.10	121.14	125.19	129.24	133.29
7Q10 tw El. 975	40000	201000	241000	128	104.65	108.82	112.99	117.16	121.33	125.50	129.67	133.84
Tw El. 980	40000	255000	295000	132	104.34	108.66	112.98	117.31	121.63	125.95	130.27	134.59
Tw El. 985	40000	290000	330000	136	104.36	108.76	113.15	117.55	121.94	126.33	130.73	135.12

These data are shown graphically in figure 66.

Table 19. - TDG analysis for the deflector alternative with a 20,000 ft^3/s transfer from joint operation with Chief Joseph Dam.

4 blocks modified

Transfer 20kcfs

	Qs cfs	Qpp cfs	Qt cfs	GCD spill TDG	River TDG levels for various reservoir TDG levels							
Reservoir TDG					100	105	110	115	120	125	130	135
Min. tw El. 958	30000	45000	75000	124	109.60	112.60	115.60	118.60	121.60	124.60	127.60	130.60
Normal tw range El. 962-966	30000	92000	122000	124	105.90	109.67	113.44	117.21	120.98	124.75	128.52	132.30
Max. tw range El. 966-970	30000	138000	168000	125	104.46	108.57	112.68	116.79	120.89	125.00	129.11	133.21
Tw El. 970-972	30000	180000	210000	126	103.71	108.00	112.29	116.57	120.86	125.14	129.43	133.71
7Q10 tw El. 975	30000	211000	241000	128	103.49	107.86	112.24	116.62	121.00	125.37	129.75	134.13
Tw El. 980	30000	265000	295000	132	103.25	107.75	112.24	116.73	121.22	125.71	130.20	134.69
Tw El. 985	30000	300000	330000	136	103.27	107.82	112.36	116.91	121.45	126.00	130.55	135.09

These data are shown graphically in figure 67.

Table 20. - TDG analysis for the forebay pipe with cascade alternative with a 10,000 ft^3/s transfer for a unit discharge of 48.8 $ft^3/s/ft$ from joint operation with Chief Joseph Dam.

	Qs cfs	Qpp cfs	Qt cfs	%TDG spill	River TDG levels for various reservoir TDG levels							
Reservoir TDG					100	105	110	115	120	125	130	135
Min. tw El. 958	40000	35000	75000	122	111.73	114.07	116.4	118.73	121.07	123.4	125.73	128.07
Normal tw range El. 962-966	40000	82000	122000	122	107.21	110.57	113.93	117.3	120.66	124.02	127.38	130.74
Max. tw range El. 966-970	40000	128000	168000	122	105.24	109.05	112.86	116.67	120.48	124.29	128.1	131.9
Tw El. 970-972	40000	170000	210000	122	104.19	108.24	112.29	116.33	120.38	124.43	128.48	132.52
7Q10 tw El. 975	40000	201000	241000	122	103.65	107.82	111.99	116.16	120.33	124.5	128.67	132.84
Tw El. 980	40000	255000	295000	122	102.98	107.31	111.63	115.95	120.27	124.59	128.92	133.24
Tw El. 985	40000	290000	330000	122	102.67	107.06	111.45	115.85	120.24	124.64	129.03	133.42

These data are shown graphically in figure 68.

Table 21.- TDG analysis for the forebay pipe with cascade alternative with a 20, 000 ft^3/s transfer for a unit discharge of 36.6 $ft^3/s/ft$ from joint operation with Chief Joseph Dam.

	Qs cfs	Qpp cfs	Qt cfs	GCD spill TDG	River TDG levels for various reservoir TDG levels							
Reservoir TDG					100	105	110	115	120	125	130	135
Min. tw El. 958	30000	45000	75000	116	106.40	109.40	112.40	115.40	118.40	121.40	124.40	127.40
Normal tw range El. 962-966	30000	92000	122000	116	103.93	107.70	111.48	115.25	119.02	122.79	126.56	130.33
Max. tw range El. 966-970	30000	138000	168000	116	102.86	106.96	111.07	115.18	119.29	123.39	127.50	131.61
Tw El. 970-972	30000	180000	210000	116	102.29	106.57	110.86	115.14	119.43	123.71	128.00	132.29
7Q10 tw El. 975	30000	211000	241000	116	101.99	106.37	110.75	115.12	119.50	123.88	128.26	132.63
Tw El. 980	30000	265000	295000	116	101.63	106.12	110.61	115.10	119.59	124.08	128.58	133.07
Tw El. 985	30000	300000	330000	116	101.45	106.00	110.55	115.09	119.64	124.18	128.73	133.27

These data are shown graphically in figure 69.

Summary of Joint Operation TDG Benefit

TDG production and mixing with power releases for the existing outlet works flow condition and each alternative has been investigated using two methods of approach. The second approach, with a spill reduction amount of 20,000 ft^3/s for a total spill of 30,000 ft^3/s has been chosen for further investigation of power transfer benefits. Two reservoir TDG levels of 110 and 125 percent were chosen as examples. The 110 percent level to show the maximum benefits under low reservoir TDG levels, and the 125 percent to show the effects for higher initial reservoir TDG levels. The dilution effect of the power releases plays an important role by increasing the weighted average of the reservoir TDG levels as the spill volume is decreased. As the TDG levels in the reservoir improve, transferring power to Grand Coulee becomes more valuable.

Of course, transferring power to reduce spill under the existing condition of outlet works gas production would be helpful because the TDG production is high under the current spill conditions and there would be no immediate capital investment. Figure 70 shows the existing outlet works operation comparison under joint operation. Only transferring flow of 20,000 ft^3/s, will improve TDG levels over those produced by the existing outlet works flow conditions by almost 7 percent at a reservoir TDG level of 110 percent to about 5 percent saturation at a reservoir level of 125 percent.

Figures 71-73 show that the benefit of power transfer will vary for the existing condition and the proposed structural alternative, depending upon the alternative, and the reservoir TDG level. The benefit of the extend and cover alternative will vary with the TDG level of the reservoir, figure 71. The extend and cover alternative will generate higher TDG levels than the power release when the reservoir levels are low (below 120 percent) and power transfer would be beneficial. At reservoir TDG levels above 119 percent, the extend and cover alternative will produce combined river TDG levels equal to the power release and power transfer would not play a role in TDG reduction. The deflector alternative will generate higher TDG levels than the power release for most reservoir TDG levels, figure 72. Therefore, transfer of power would be minimally beneficial. A reduction in cost would occur with power transfer and reduced spill for both the extend and cover and the deflector alternatives. With the forebay cascade alternative there is always an improved TDG condition with reduced spill, figure 73. However, there is no associated decrease in cost with the forebay pipe with cascade alternative.

A direct comparison of the TDG levels and cost is given in Table 22 for the existing condition, and each alternative with and without power transfer for a spill of 30,000 ft^3/s for the example TDG levels chosen. The TDG values shown in Table 22 were compiled from Tables 1, 2, 4, 7, 15, 17, 19 and 21. The TDG levels from the existing outlet works flow can be reduced by about 7 percent with reservoir TDG of 110 percent and 5 percent with the reservoir at 125 percent and the 20,000 ft^3/s transfer under joint operation. The greatest difference between the TDG of the existing condition and the full alternative is for the extend and cover alternative at 15 percent saturation with the reservoir at 110 percent. The greatest overall difference in TDG occurs when comparing the TDG level of the existing flow condition with the reservoir at 110 percent (TDGex) to that of the forebay cascade alternative with transfer (TDGt) and reduced spill producing a difference of 16 percent saturation. There are only small differences between the TDG of the full alternative (TDGf) and the spill transfer (TDGt), except for the almost 3 percent decrease when using the forebay cascade alternative with the reservoir TDG level at 110 percent. Table 22 also shows the cost benefit for the extend and cover and the deflector alternatives in transferring power by allowing a reduction in the number of blocks or outlet pairs that would be modified. The cost benefit, however, must be weighed against the potential for improvement in the upstream lake TDG levels and the overall TDG benefit. There is no cost benefit with the forebay pipe with cascade alternative and minimal TDG benefit when the reservoir TDG levels are low. Transferring power will benefit the existing outlet works operation with no additional capital cost.

Table 22. - Comparison of the TDG characteristics and cost for reservoir TDG levels of 110 and 120 percent for the existing condition, construction of the full alternative, and the effect of joint operation.

7Q10=241,000 cfs		River TDG with reservoir at 110%								
	existing condition	full alternative	difference TDGex-TDGf	flow transfer	difference TDGex-TDGt	difference TDGf-TDGt	full alterative	cost ($M)	modification	cost ($M)
	50,000 cfs	50,000 cfs		30,000 cfs			no. of blocks		no. of blocks	
existing outlets	126.6			119.96	6.64					
extend and cover		111.87	14.73	111.12	15.48	0.75	8	96.2	5	72.8
deflectors		113.73	12.87	112.24	14.36	1.49	6	15.6	4	11.3
forebay pipe with cascade		113.57	13.03	110.86	15.74	2.71	full	437.0	full	437.0
		River TDG with reservoir at 125%								
	existing condition	full alternative	difference TDGex-TDGf	flow transfer	difference TDGex-TDGt	difference TDGf-TDGt				
	50,000 cfs	50,000 cfs		30,000 cfs						
existing outlets	138.49			133.09	5.4					
extend and cover		125.00	13.49	125.00	13.49	0.00				
deflectors		125.62	12.87	125.37	13.12	0.25				
forebay pipe with cascade		125.00	13.49	123.88	14.61	1.12				

These data are shown graphically in figures 70, 71, 72 and 73.

Conclusions

This report provides information regarding the TDG performance and structural cost for feasibility-level designs of three proposed modifications at Grand Coulee Dam for total dissolved gas abatement. There is also a discussion of the impact of joint operation between Grand Coulee and Chief Joseph Dams. This information can be used in a system-wide evaluation of gas abatement measures for the upper, middle and lower Columbia and Snake Rivers.

First, the resulting TDG levels in the river below Grand Coulee Dam for each alternative are summarized. The alternatives are compared to the existing outlet works flow conditions over the range of reservoir TDG levels from 100 to 135 percent saturation. The deflector alternative is the least expensive to construct, but will provide the least TDG benefit of about a 12.9 percent saturation reduction. The forebay pipe with cascade alternative is the most expensive alternative to construct and will provide additional TDG benefit of about a 13.5 percent saturation reduction. The extend and cover alternative is the second least expensive, but provides the most TDG benefit of about 14.7 to 11.4 percent saturation reduction as the reservoir TDG level increases to 135 percent. Therefore, the extend and cover alternative provides the most TDG benefit until the reservoir TDG levels increase at which point the alternatives all provide about the same level of TDG reduction. A combined ranking of the cost and the TDG effectiveness of each alternative was developed as the sum of the cost and TDG effectiveness. The extend and cover alternative has the highest ranking as per Table 9 of the Comparisons.

Second, the impact of joint operation between Grand Coulee and Chief Joseph Dam was investigated and compared to the existing outlet works flow conditions and each alternative using two methods of analysis. Reported here will be the analysis that reduced spill by a constant 20,000 ft^3/s flow with reservoir TDG levels of 110 and 125 percent saturation. This transfer flow produced a 5 to 7 percent TDG benefit for the existing outlet works flow condition without structurally modifying the dam.

The overall TDG benefit in the river downstream from Grand Coulee for each alternative would be increased only by 1 to 2 percent saturation with joint operation. This percent TDG benefit is fairly insignificant. The major benefit of flow transfer would be in reducing the cost from constructing the full alternative to a portion of the extend and cover or deflector alternative with flow transfer. Flow transfer would allow the number of outlets modified to be reduced, thus producing a cost savings of $4.3 million for the deflector and $23.4 million for the extend and cover alternative, respectively. There would be no cost benefit with the forebay cascade alternative. This would also potentially increase the risk that flows would be exceeded and leave no room for mechanical problems, etc that could negate benefits. Joint operation investigations do not change the relative ranking of the alternatives.

These designs are presented for consideration in the effort reduce the TDG in the total Columbia Basin system-wide study. These designs may change somewhat in the final design phase, especially the forebay pipe with cascade, alternative 5. Additional two- and three-dimensional studies may be required to prepare the final design and evaluate other impacts, i.e. the riverbank protection downstream of Grand Coulee Dam when subjected to higher energy flows from the deflectors. The design discharge for this report was 50,000 ft^3/s and TDG benefits may need additional evaluation if a different spill flow is ultimately selected. Joint operation may also change the extent of the proposed structural modifications. The costs for extend and cover and the deflector alternatives have been estimated per block to allow comparison of various level of modification.

References

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[4] “Chief Joseph Dam, Gas Abatement Study, General Reevaluation Study”, US Army Corps of Engineers, April 2000.

[5] REC-ERC-77-14, “HydraulicTests and Development of Multijet Sleeve Valves”, U.S. Bureau of Reclamation, Technical Service Center, Denver CO, December 1977.

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[7] Cain, James D., “Design of Spillway Deflectors for Ice Harbor Dam to Reduce Supersaturated Dissolved Gas Levels Downstream”, IAHR Conference, pp. 607-611, San Francisco CA, August 1997.

[8] Falvey, Henry T., “Cavitation in Chutes and Spillways”, U.S. Bureau of Reclamation, Engineering Monograph No. 42, Denver, CO, 1990.

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[10] “Design of Small Dams, third edition”, U.S. Bureau of Reclamation, GPO, Denver, CO, 1987.

[11] Technical Memorandum No. 50, “John Day Lock and Dam Spillway Flow Deflectors”, Design Memorandum #50, U.S. Army Corps of Engineers, Portland District, August 1996.

[12] AFT Fathom Pipe Flow Analysis System for Windows, Applied Flow Technology Corporation, copyright 1994, U.S.

[13] Matos, Jorge, “Hydraulic Design of Stepped Spillways Over RCC Dams”, Hydraulics of Stepped Spillways, Minor and Hager, A.A. Balkema, Rotterdam, pp. 187-194, March 2000.

[14] Schneider, Michael L., Wilhelms, Steven C., “Ice Harbor Raised Tailrace - Estimated TDG Saturation”, Memorandum for record, US Army Corps of Engineers, Coastal and Hydraulics Laboratory, October 26, 1998.

Figures

Figure 1. **-** Grand Coulee Dam with spillway and powerplant operating.

Figure 2. **-** Grand Coulee Dam, close up of some of the outlet works operating.

Figure 3. - Overall view of Grand Coulee Dam and Third Powerplant. Notice the north service yard adjacent to the Third Powerplant.
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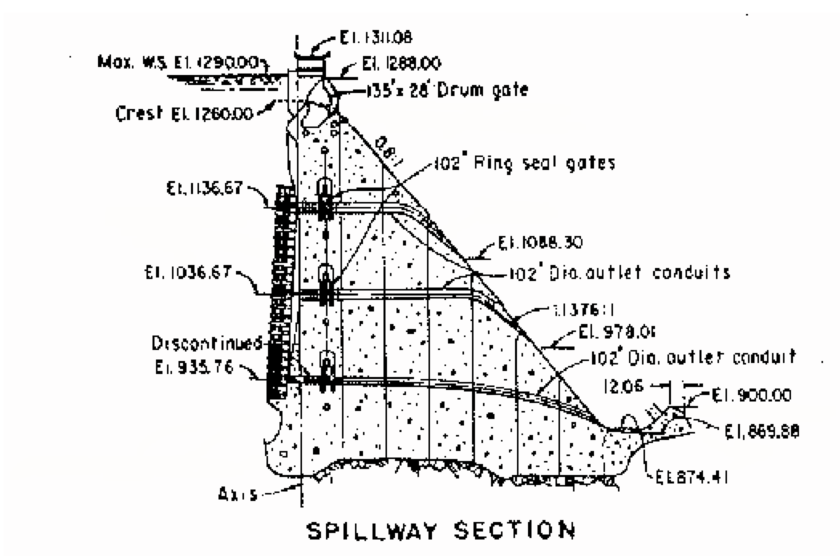


Figure 4. - Section of the Grand Coulee spillway showing the locations of the spillway crest and drum gates, the three tiers of outlet works, and the roller bucket energy dissipater.

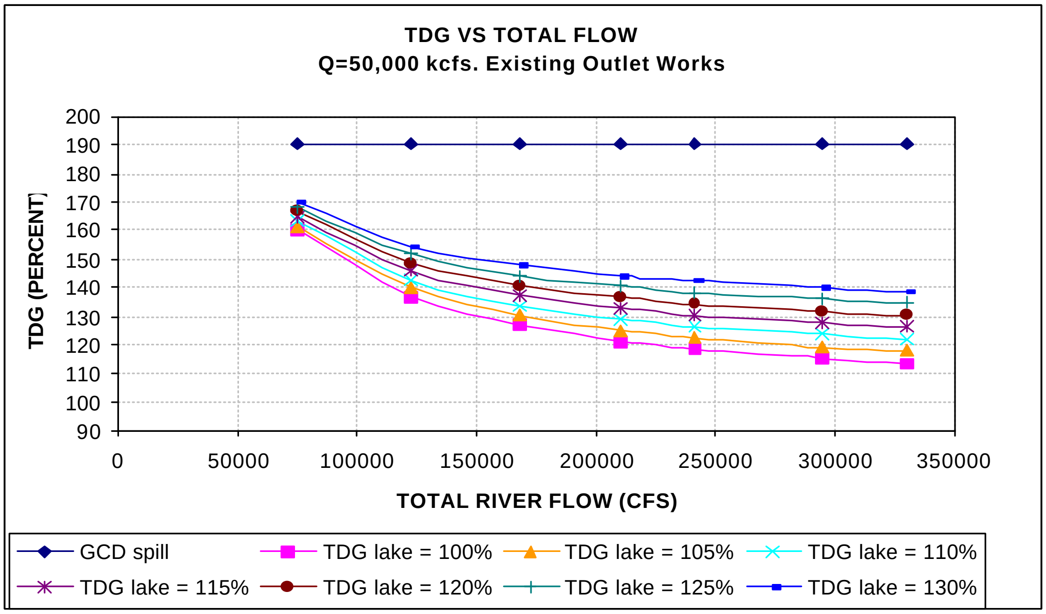


Figure 5. - Existing outlet works TDG production with the 50,000 ft^3/s design discharge passed and then mixed with various lake TDG levels through powerplant releases.

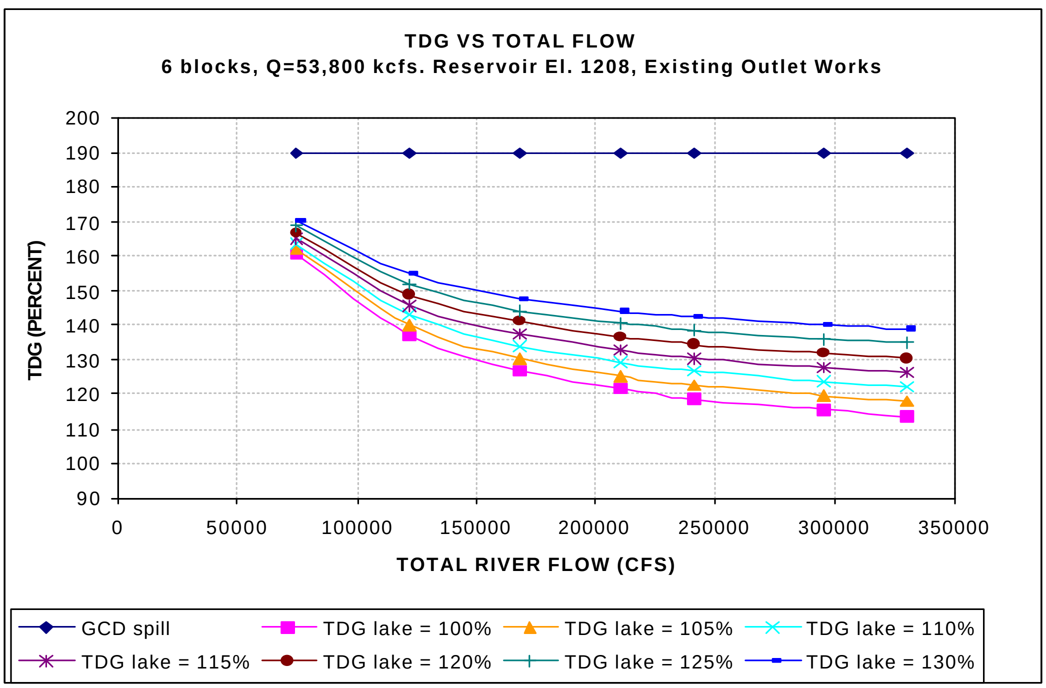


Figure 6. - Existing outlet works TDG production with the design discharge passed by 6 pairs of outlets at reservoir El. 1208. The outlet spill is mixed with various lake TDG levels through powerplant releases.

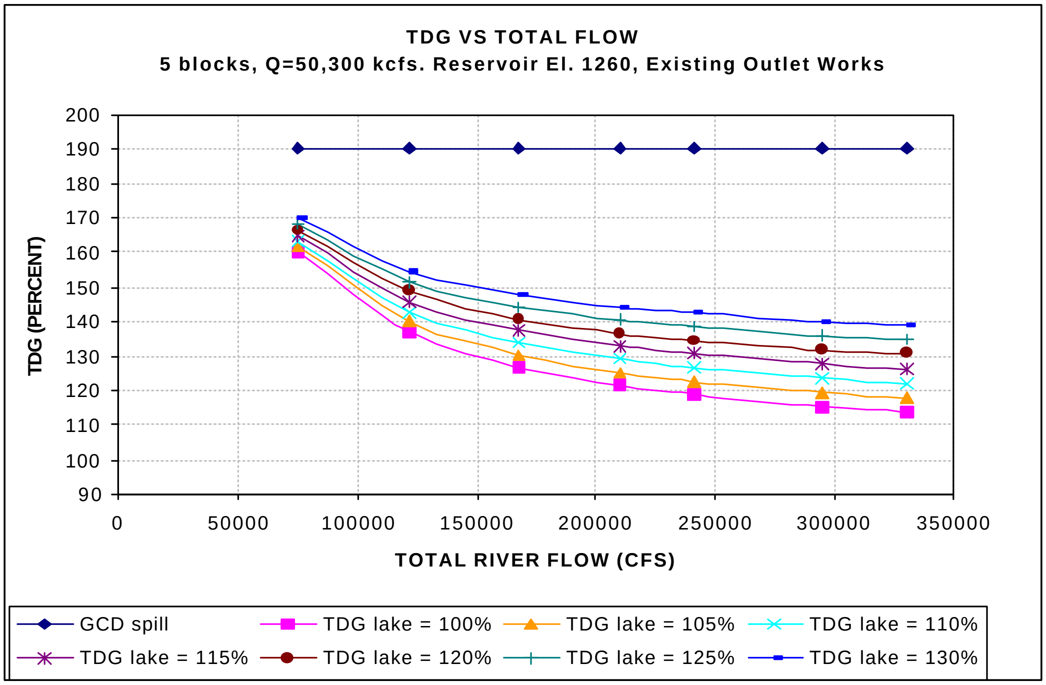


Figure 7. - Existing outlet works TDG production with the design discharge passed by 5 pairs of outlets at reservoir El. 1260. The outlet spill is mixed with various lake TDG levels through powerplant releases.

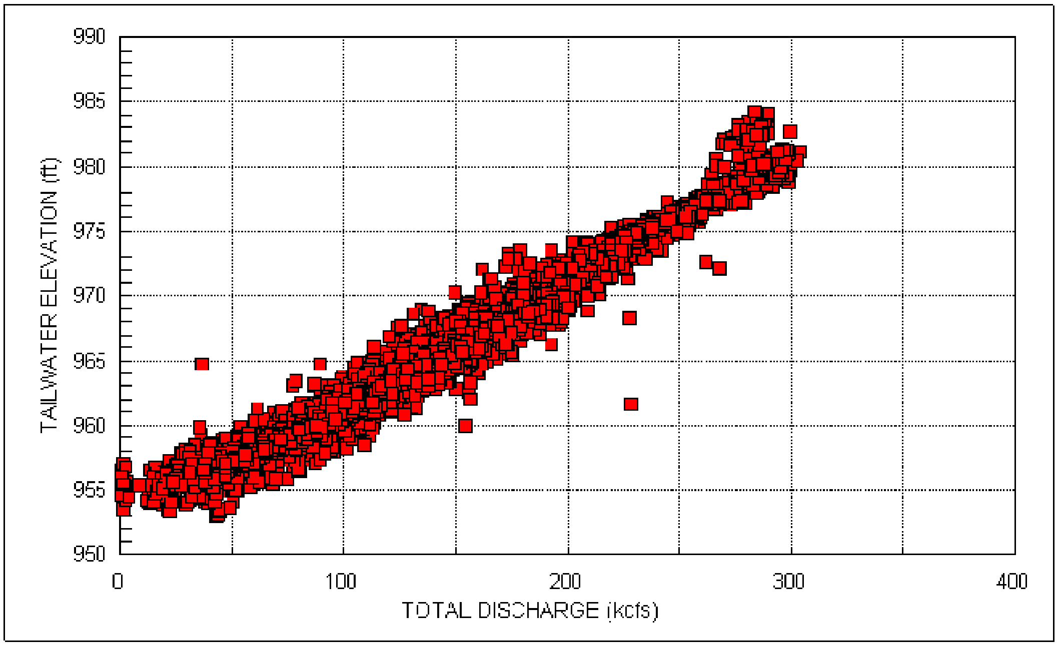


Figure 8. - Tailwater data from the gage located at the highway bridge below Grand Coulee Dam for 1997.

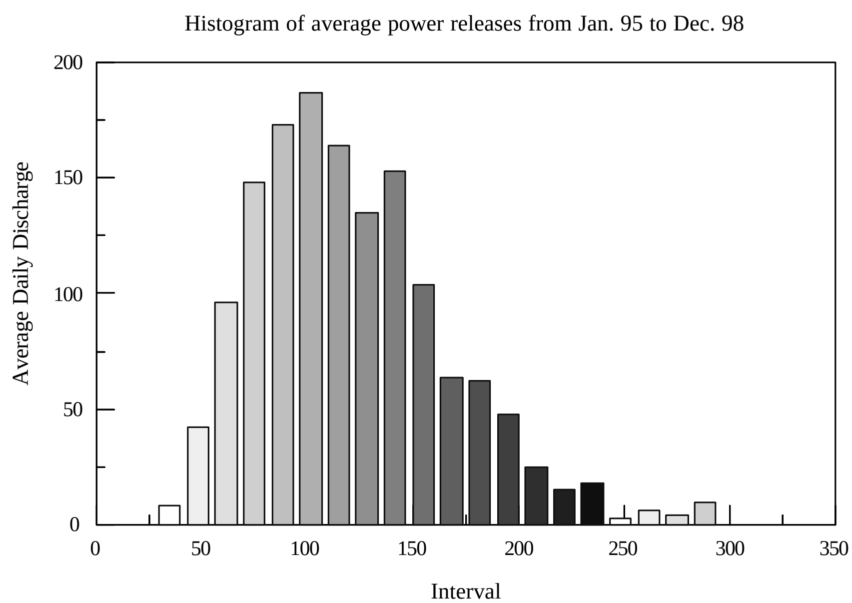
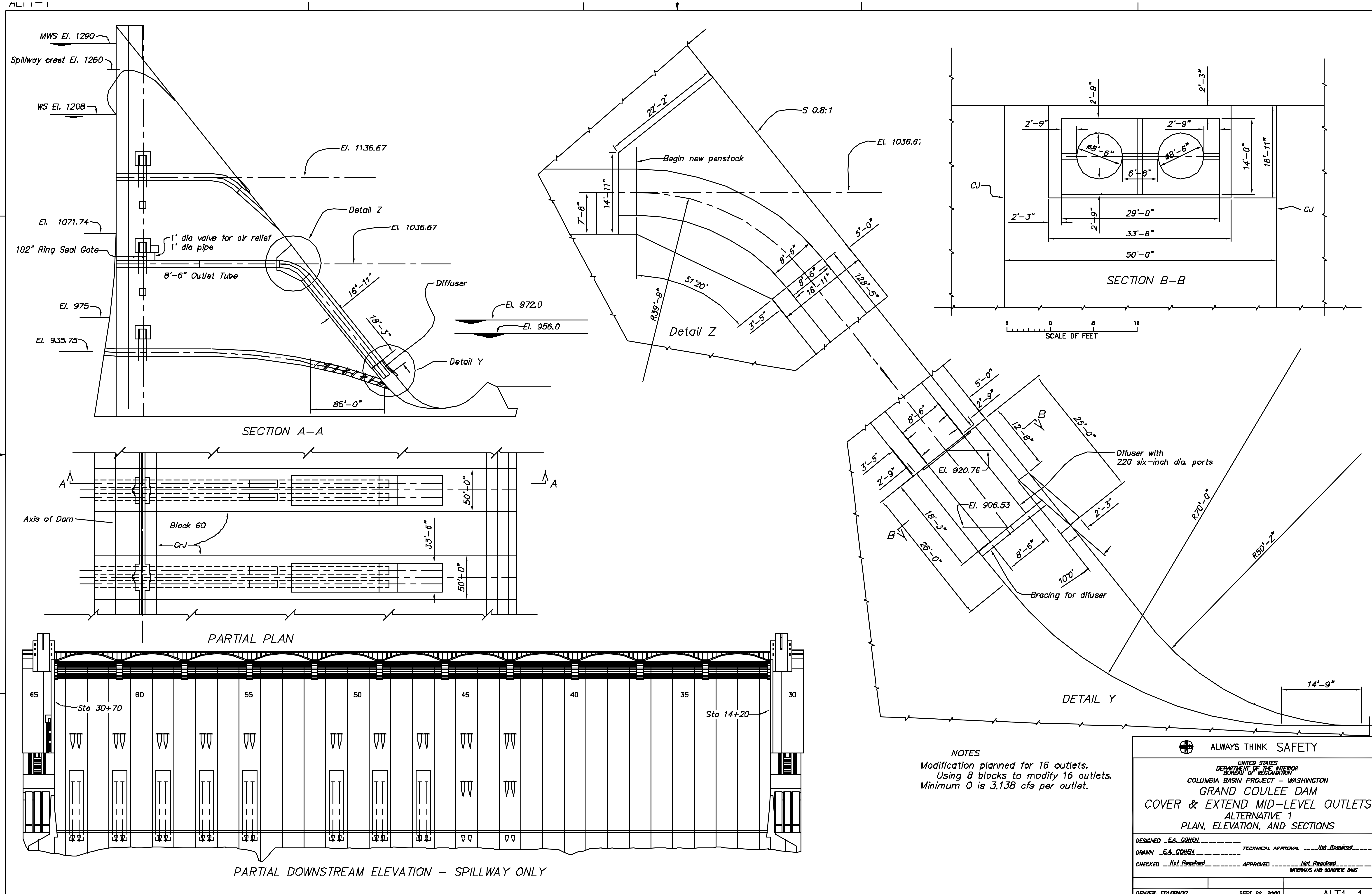


Figure 9. - Histogram of the average powerplant releases from the COE web site for January 1995 to December 1998 for use in evaluating a "normal" tailwater range for the deflector alternative design.



Drawing of the extend and cover alternative – Figure 10

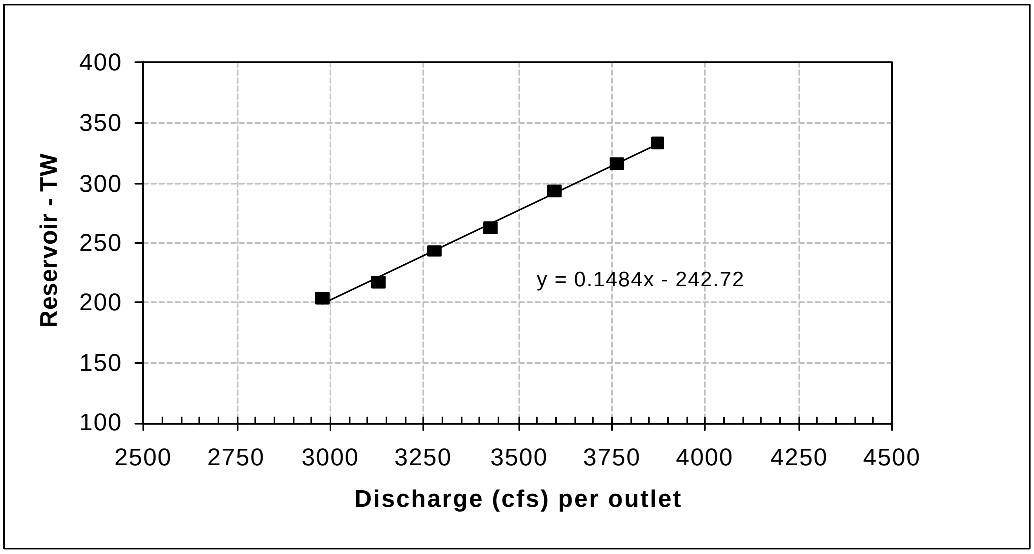


Figure 11. - Discharge rating per outlet for the 220 hole diffuser as a function of the reservoir head and tailwater depth.

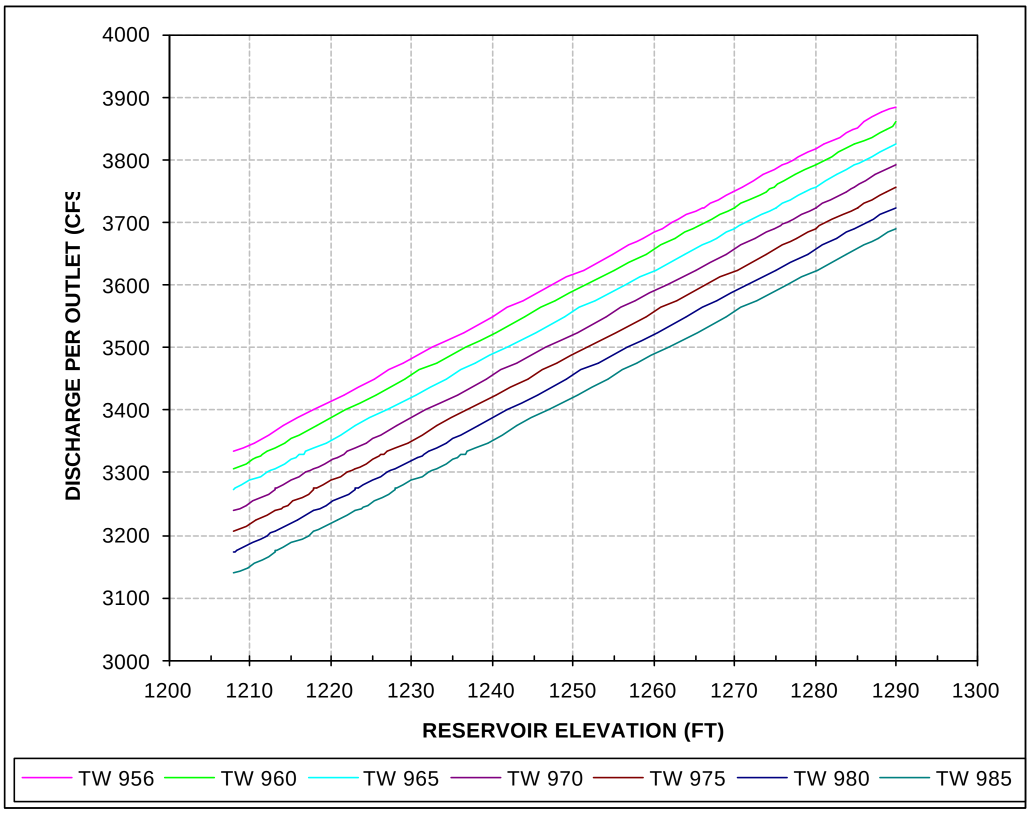


Figure 12. - Family of curves for the diffuser operation as a function of reservoir and tailwater elevation.

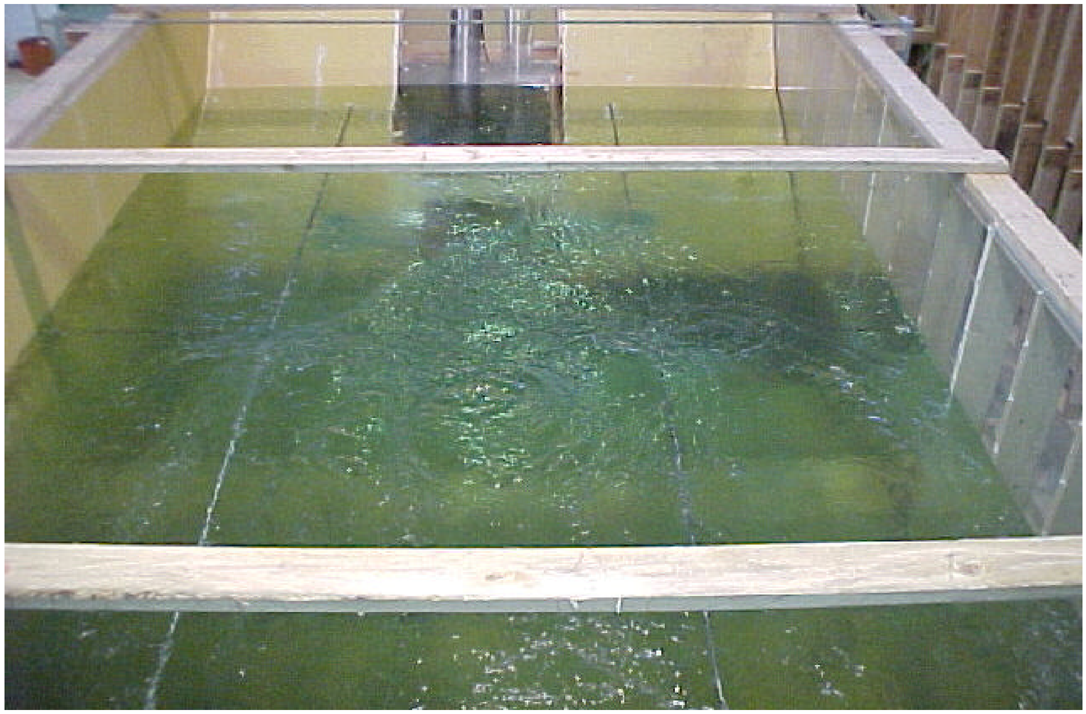


Figure 13. **-** Surface flow condition for the 220 hole diffuser with reservoir El 1260 and tailwater El 970.

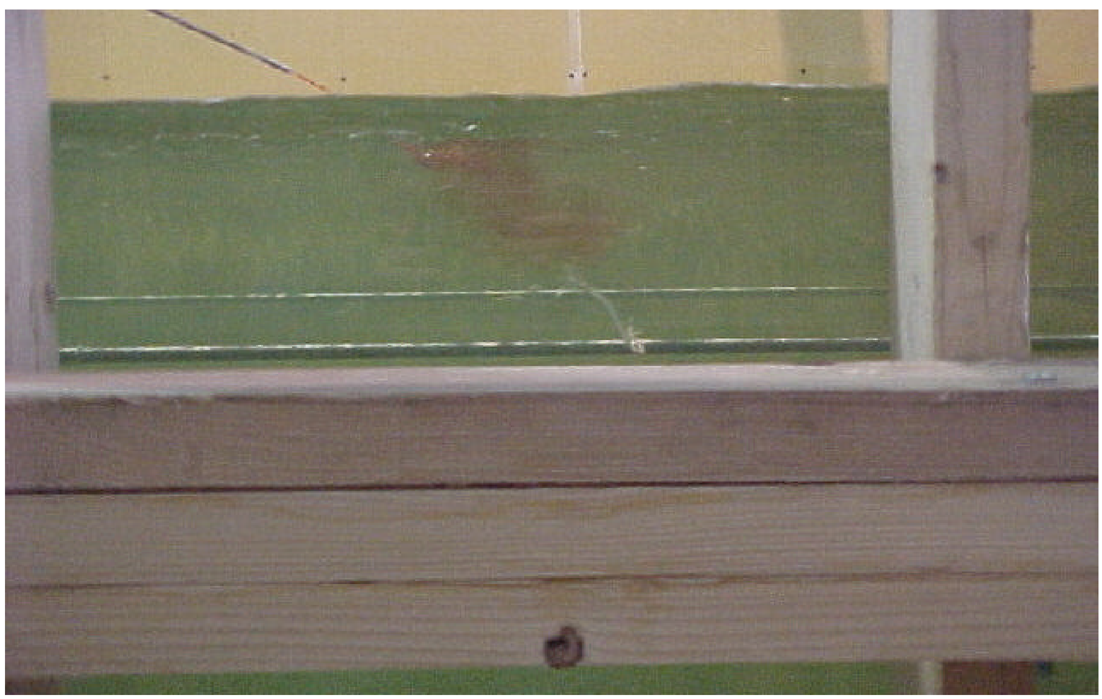


Figure 14. **-** Side view of outlet works operations with the diffuser under Res. El. 1260, tailwater 970 with flow from right to left. Minimal surface turbulence is observed. Fiber tuft indicates upward and slightly downstream flow direction. Dye indicating the same flow directions, upward and downstream.

Figure 15. **-** Side view of dye tracing for the 220 hole diffuser with res. El. 1260, tailwater 970. Note the fiber tuft indicating an upward velocity component and the dye trace indicating a downstream movement with no strong recirculation patterns of turbulence. This photograph is similar to figure 14.

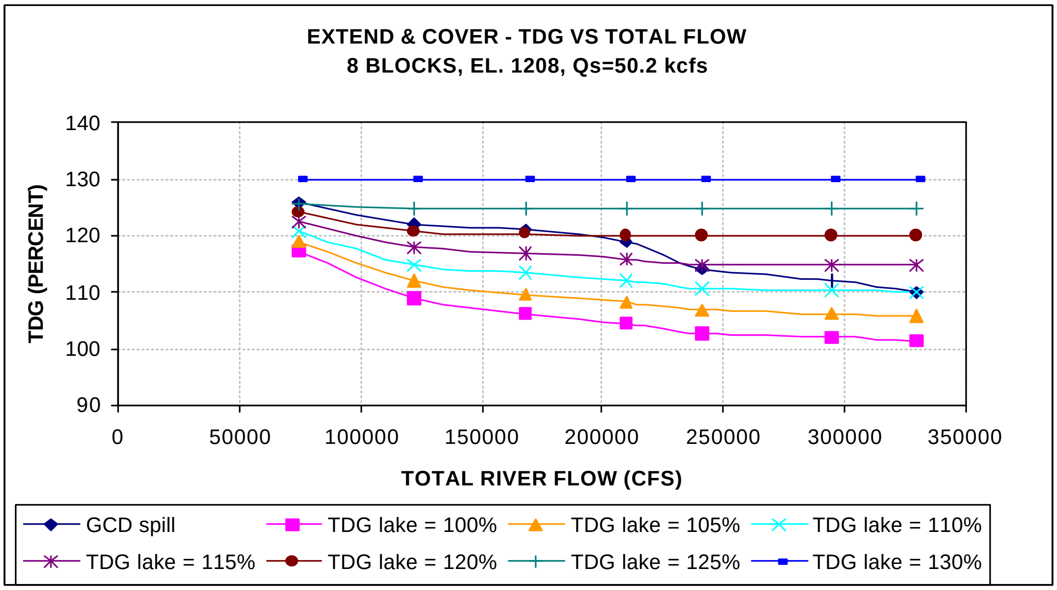


Figure 16. - TDG levels for the diffuser designed for the extend and cover alternative. The TDG of the design spill at reservoir El. 1208 alone is shown with a series of curves for total river flow depending upon various reservoir TDG levels.

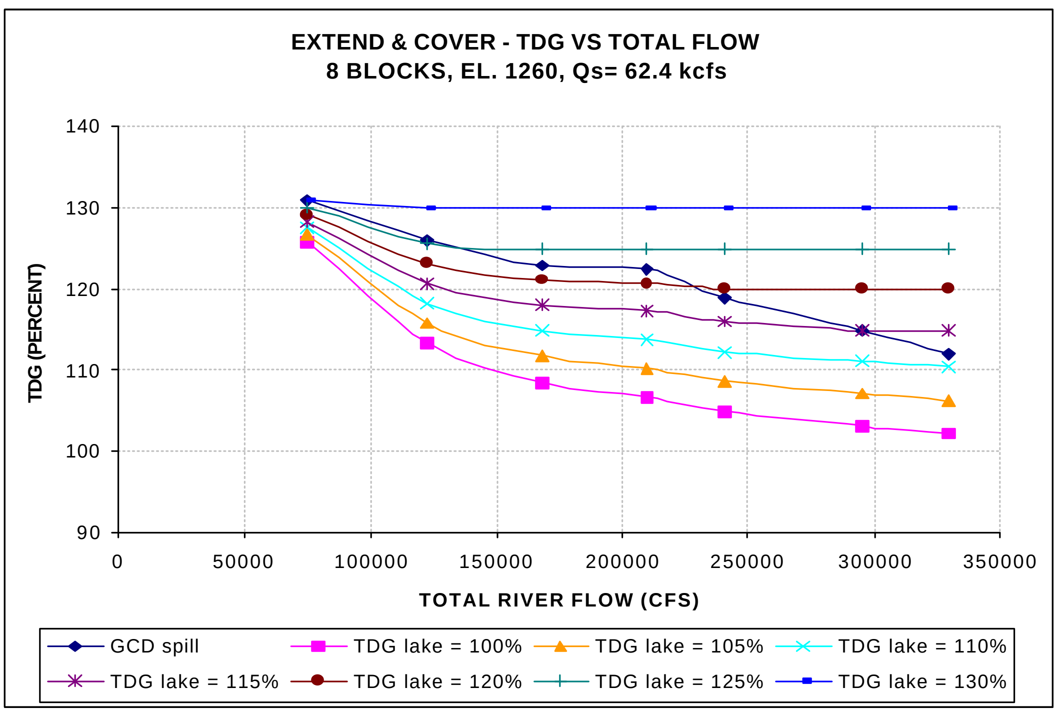
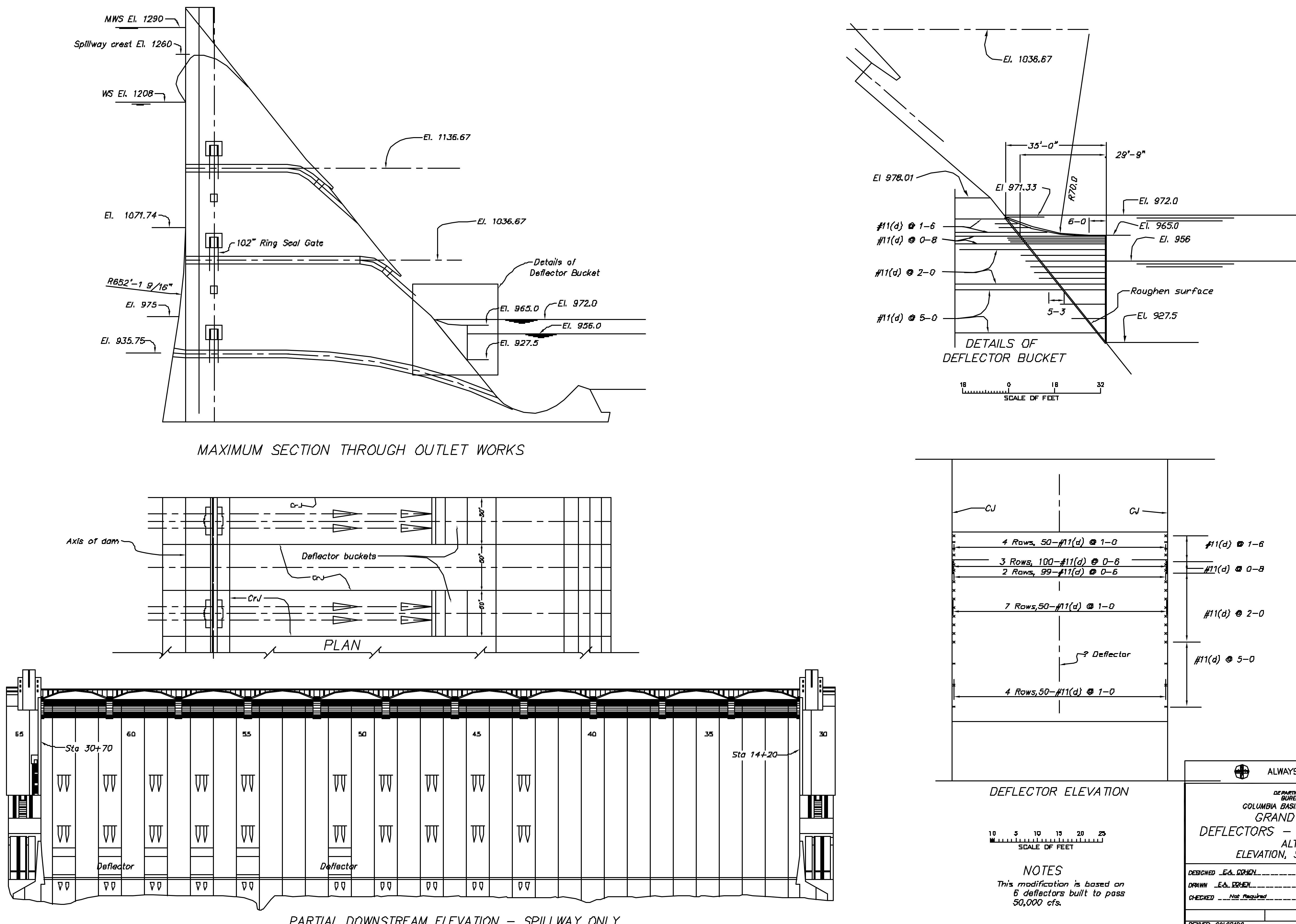


Figure 17. - TDG levels for the diffuser designed for the extend and cover alternative. The TDG of the design spill at reservoir El. 1260 alone is shown with a series of curves for total river flow depending upo various reservoir TDG levels.



NOTES
This modification is based on 6 deflectors built to pass 50,000 cfs.

ALWAYS THINK SAFETY
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT - WASHINGTON
GRAND COULEE DAM
DEFLECTORS - MIN. NO. OF OUTLETS
ALTERNATIVE 3
ELEVATION, SECTION, AND DETAIL

DESIGNED	E.A. COHEN	TECHNICAL APPROVAL	Not Required
DRAWN	E.A. COHEN		
CHECKED	Not Required	APPROVED	Not Required WATERWAYS AND CONCRETE DAMS
DENVER, COLORADO	SEPT 29, 2000	ALT3-1	

Drawing of the deflector alternative – FIGURE 18

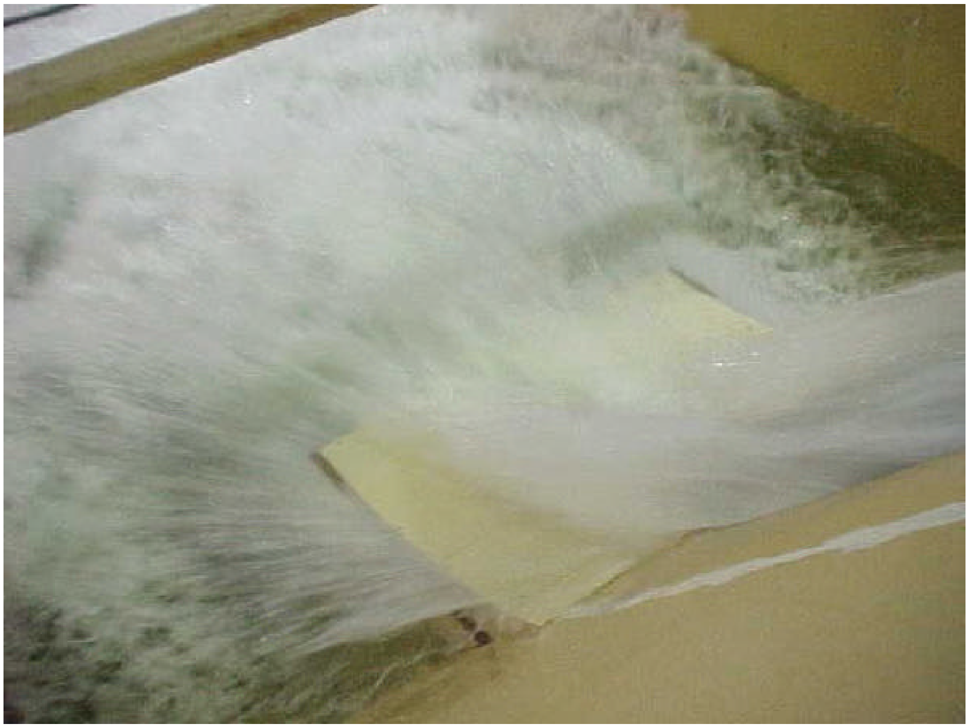


Figure 19. **-** Typical flow pattern and jet trajectory for the mid-level outlets operating at minimum reservoir El. 1208 and tailwater El. 958.

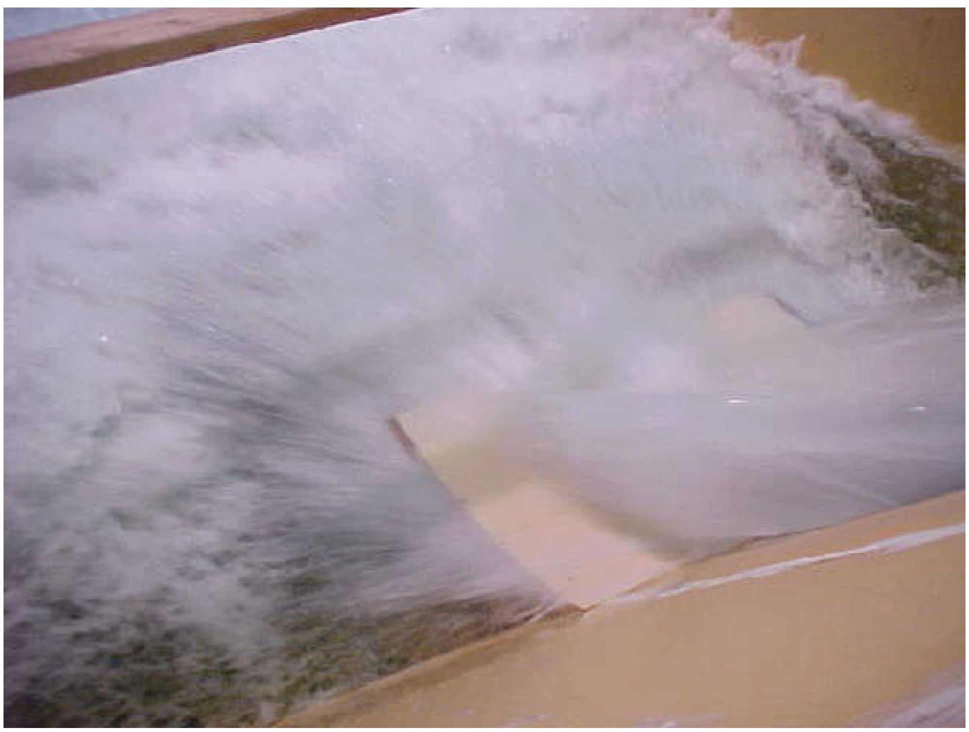


Figure 20. **-** Typical flow patterns and jet trajectory for the mid-level outlets operating at maximum reservoir El. 1290 and tailwater El. 958.

Figure 21. **-** Typical jet trajectory for the upper outlets operating at minimum reservoir El. 1208 and tailwater El. 958.

Figure 22. **-** Typical jet trajectory for the upper outlets operating at minimum reservoir El 1208 and tailwater El. 958.

Figure 23. **-** Typical jet trajectory for the upper outlets operating at maximum reservoir El. 1290 and tailwater El. 958.

Figure 24. **-** Typical jet trajectory for the upper outlets operating at maximum reservoir El. 1290 and tailwater El. 958.

Figure 25. **-** Typical jet trajectory for combined outlets operating at minimum reservoir El. 1208 and tailwater El. 958.

Figure 26. **-** Typical jet trajectory for combined outlets operating at minimum reservoir El. 1208 and tailwater El. 958.

Figure 27. **-** Typical jet trajectory for combined outlets operating at maximum reservoir El. 1290 and tailwater El. 958.

Figure 28. **-** Typical jet trajectory for combined outlets operating at maximum reservoir El. 1290 and tailwater El. 958.

Figure 29. **-** Deflector Alternative with typical combined mid- and upper-outlet works operating. Reservoir at El. 1290 and high tailwater at El. 970.

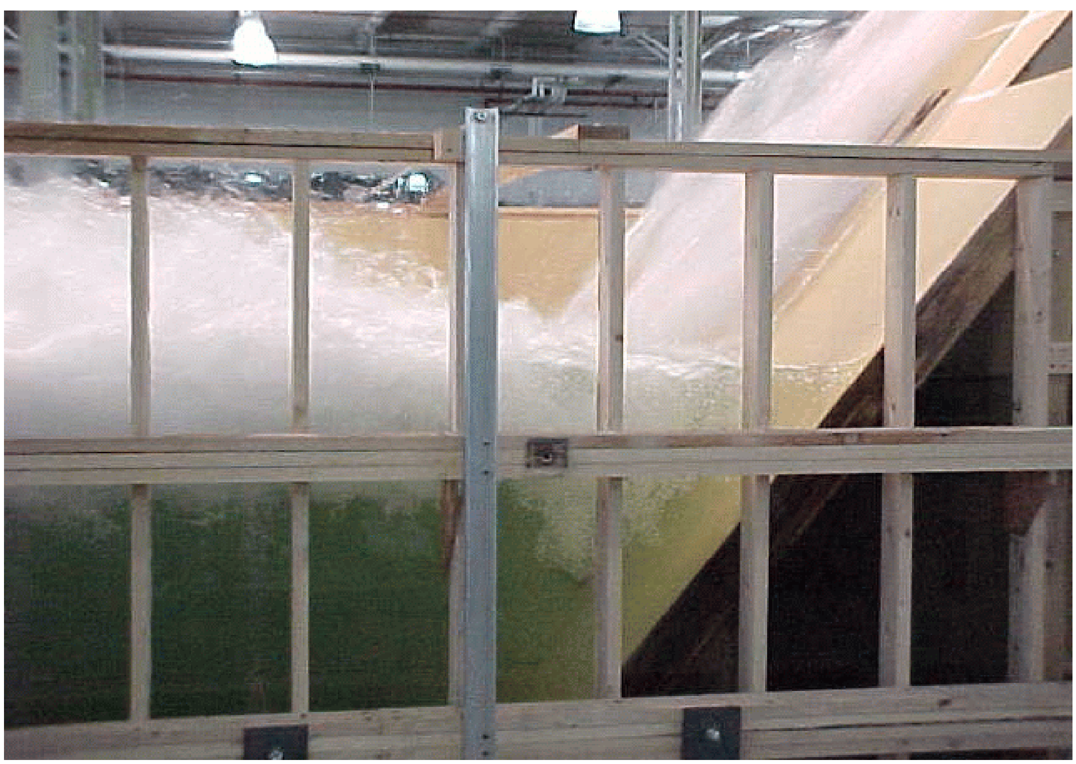


Figure 30. **-** Typical view for combined mid- and upper-outlet works operating at maximum reservoir El. 1290 and high tailwater El. 970.

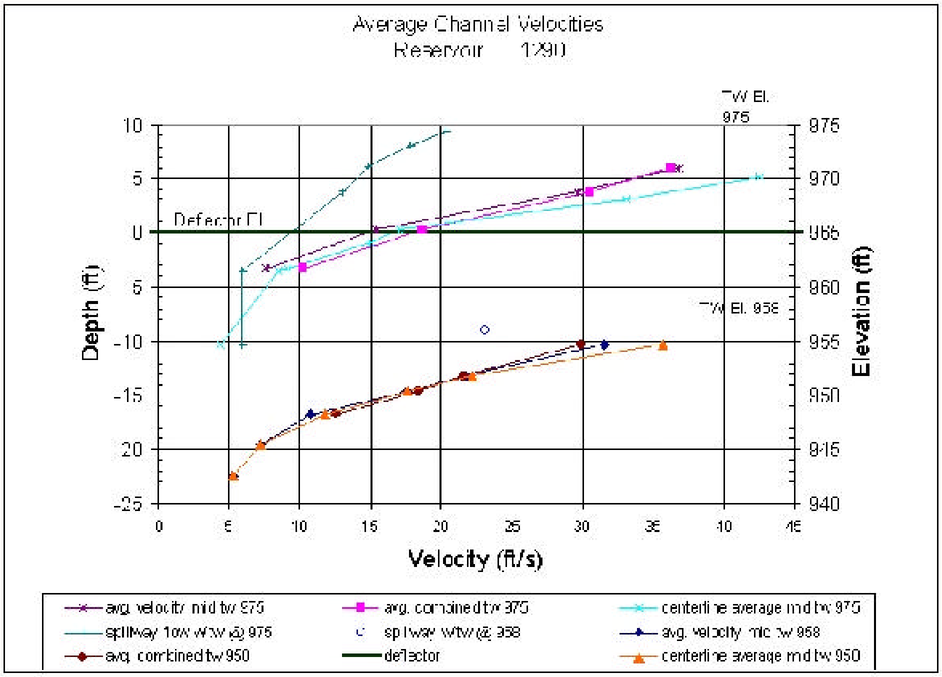


Figure 31. **-** Average velocities about 185 ft downstream from the existing flip bucket for the deflector option. The velocities are referenced to both the tailwater depths and the deflector elevation.

Figure 32. **-** Simulated spillway flow down the dam face and impacting on the deflector. Spillway discharges were more easily deflected than the outlet works discharges, behaving more similar to expected deflector performance.

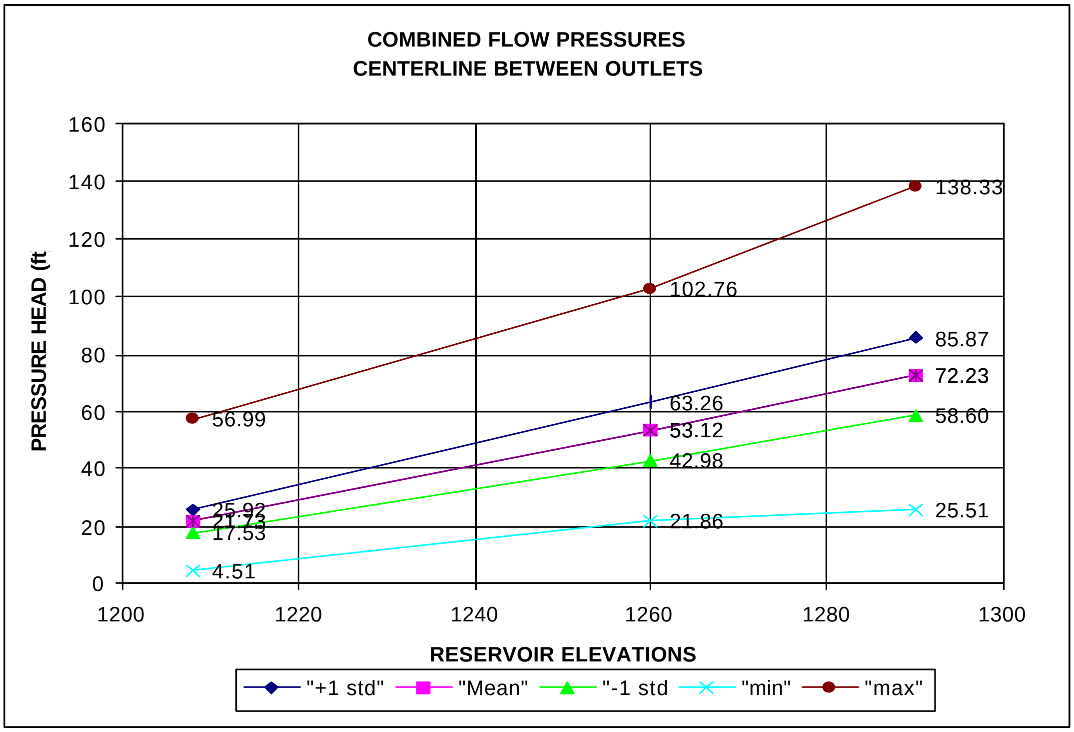


Figure 33. - Pressures measured along the centerline of the deflector between the outlets for combined flow conditions. These pressures were used for structural design.

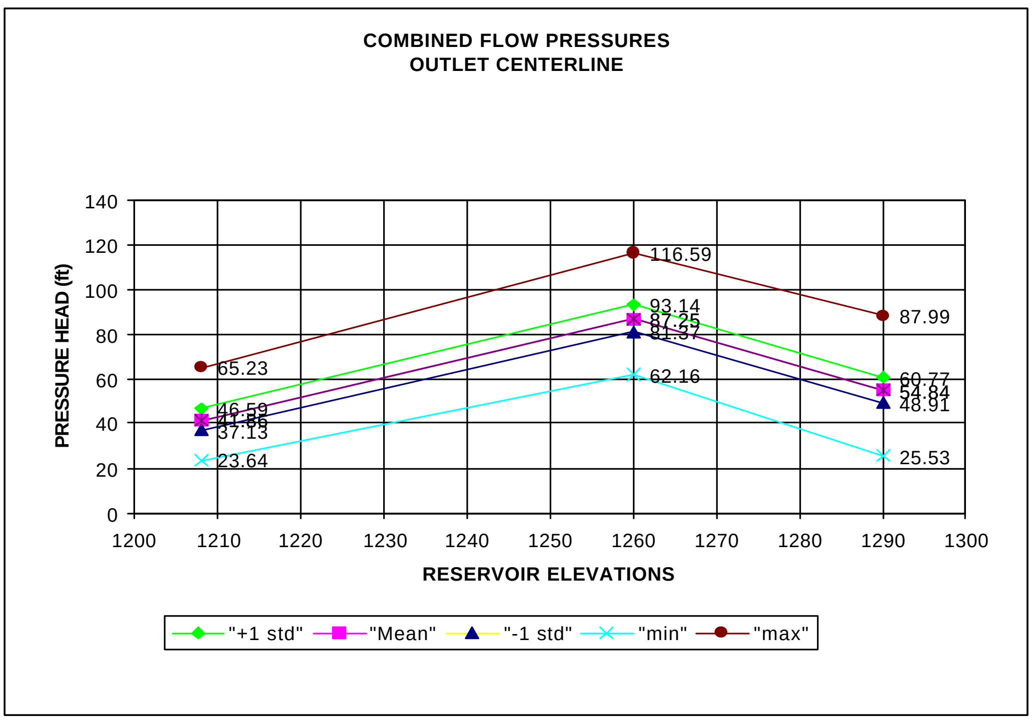


Figure 34. - Pressures measured along the centerline of the outlet over the deflector for combined flow conditions. These pressures were used for structural design.

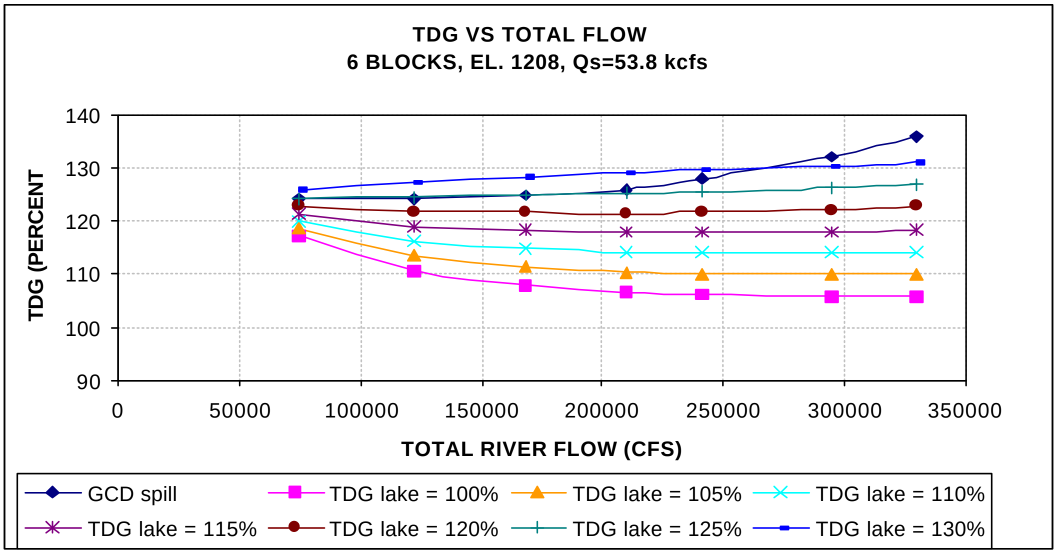


Figure 35. **-** TDG levels for the 50,000 ft^3/s design spill in the river for the deflector alternative with reservoir El. 1208 and the full range of tailwater and river flow.

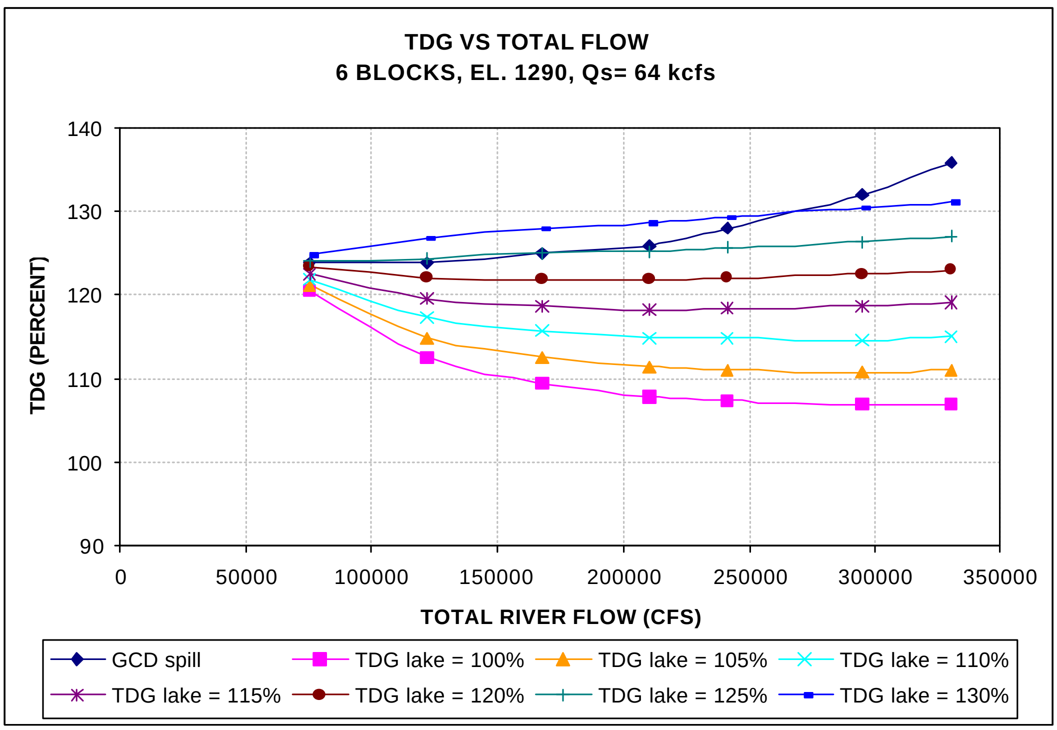
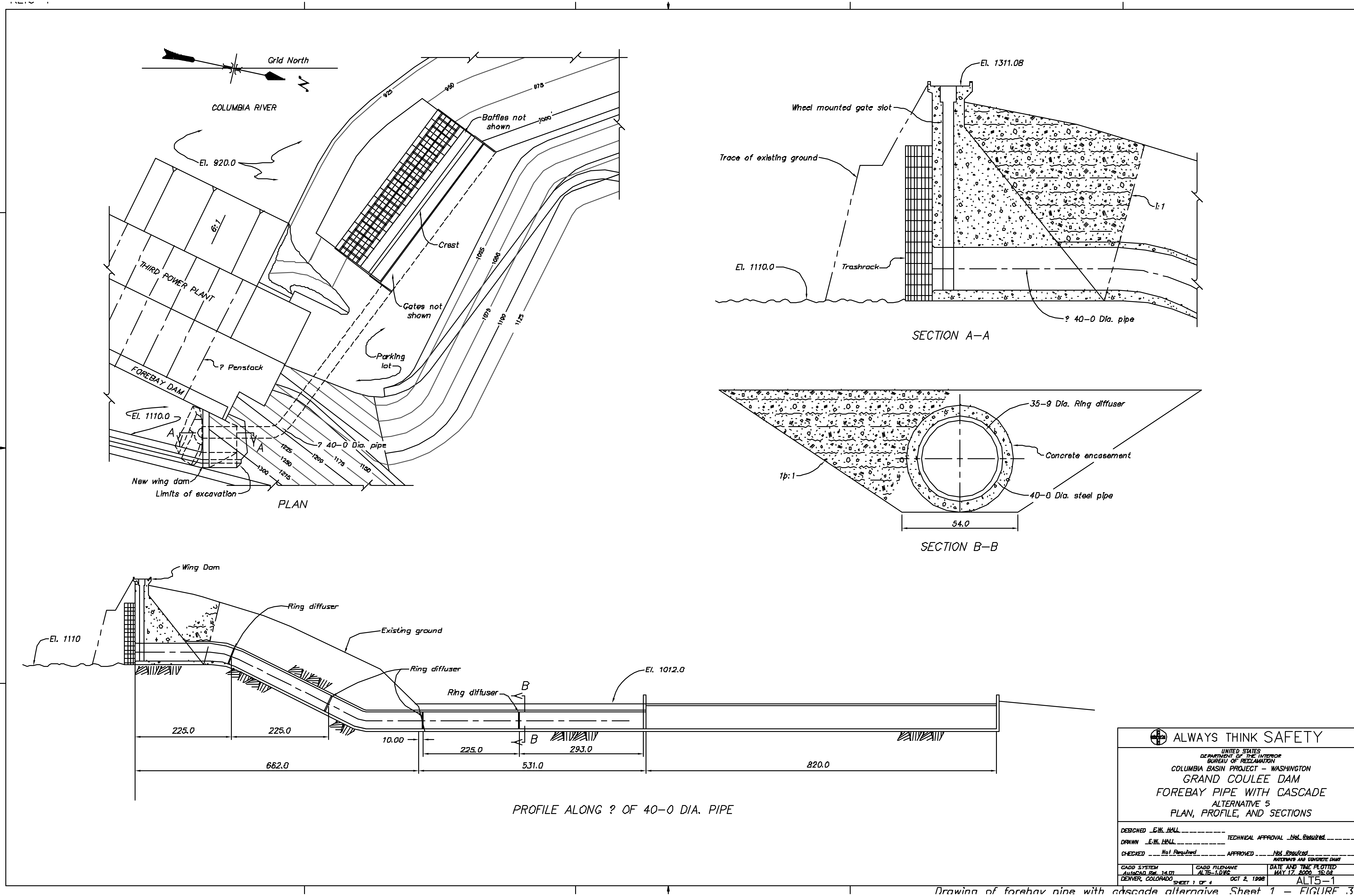
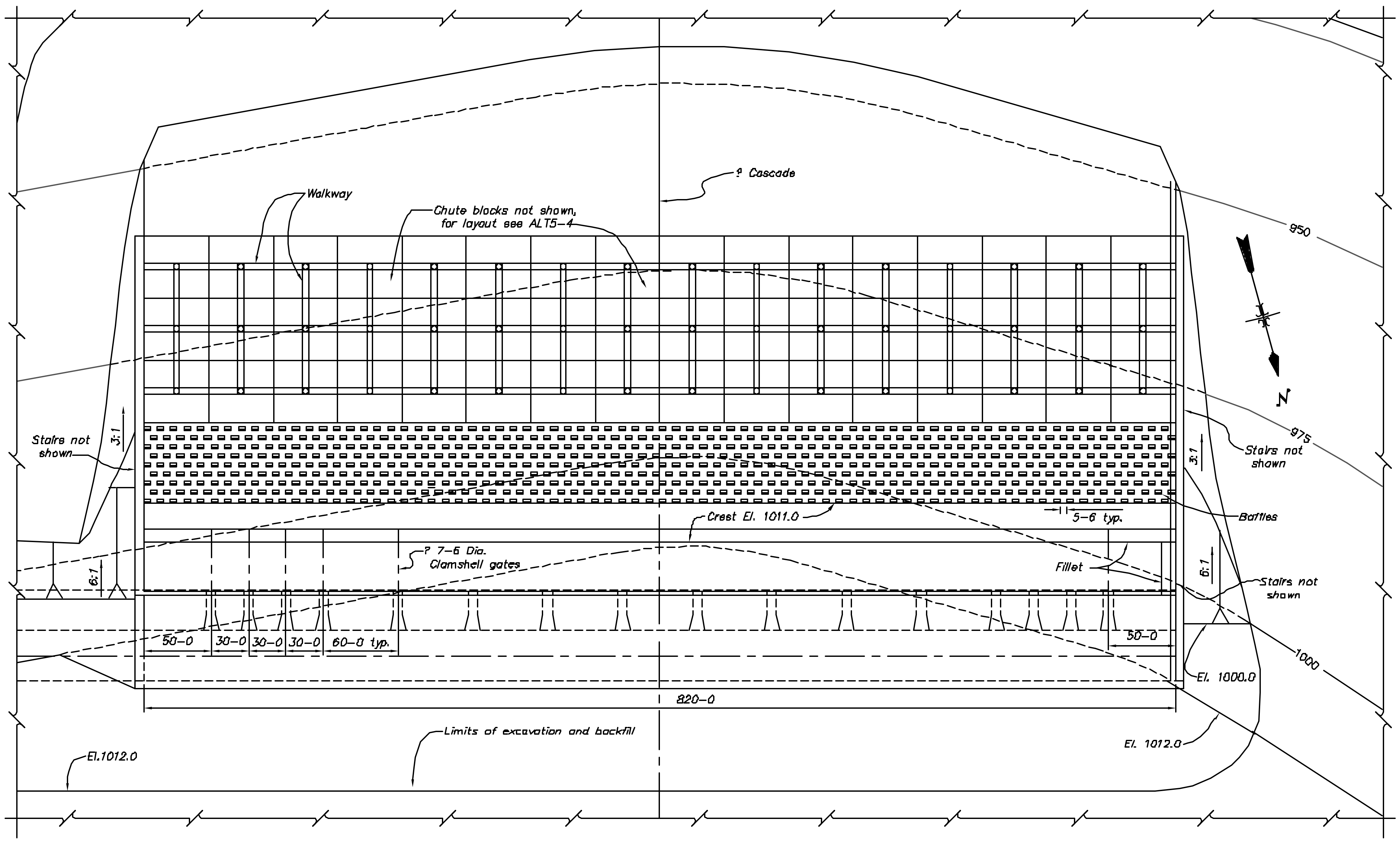


Figure 36. **-** TDG levels for the 50,000 ft^3/s design spill in the river for the deflector alternative with reservoir El. 1290 and the full range of tailwater and river flow.



Drawing of forebay pipe with cascade alternaive Sheet 1 - FIGURE 37



PARTIAL PLAN OF CASCADE

50 0 50 100 150
SCALE OF FEET

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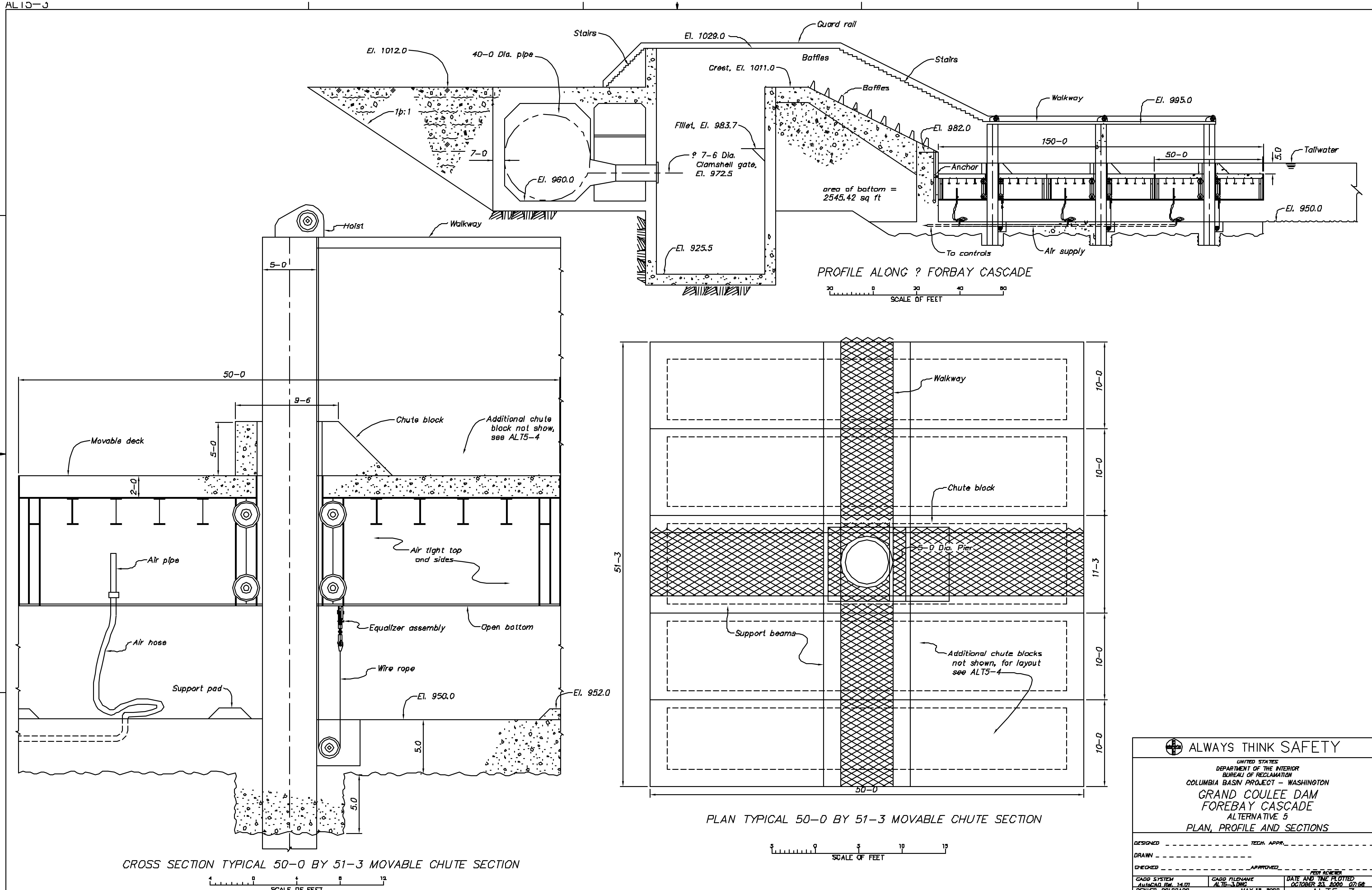
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT - WASHINGTON
GRAND COULEE DAM
FOREBAY PIPE WITH CASCADE
ALTERNATIVE 5
PLAN, PROFILE AND SECTIONS

DESIGNED E.W. HALL	
DRAWN E.W. HALL	TECHNICAL APPROVAL Not Required
CHECKED Not Required	APPROVED Not Required WATERWAYS AND CONCRETE DAMS

CADD SYSTEM AutoCAD Rel. 14.01	CADD FILENAME ALT5-2-14.DWG	DATE AND TIME PLOTTED MAY 23, 2000 09:13
DENVER, COLORADO SHEET 2 OF 4	OCT 2, 1998	ALT5-2

Drawing of forebay pipe with cascade alternative – Sheet 2 – FIGURE 38



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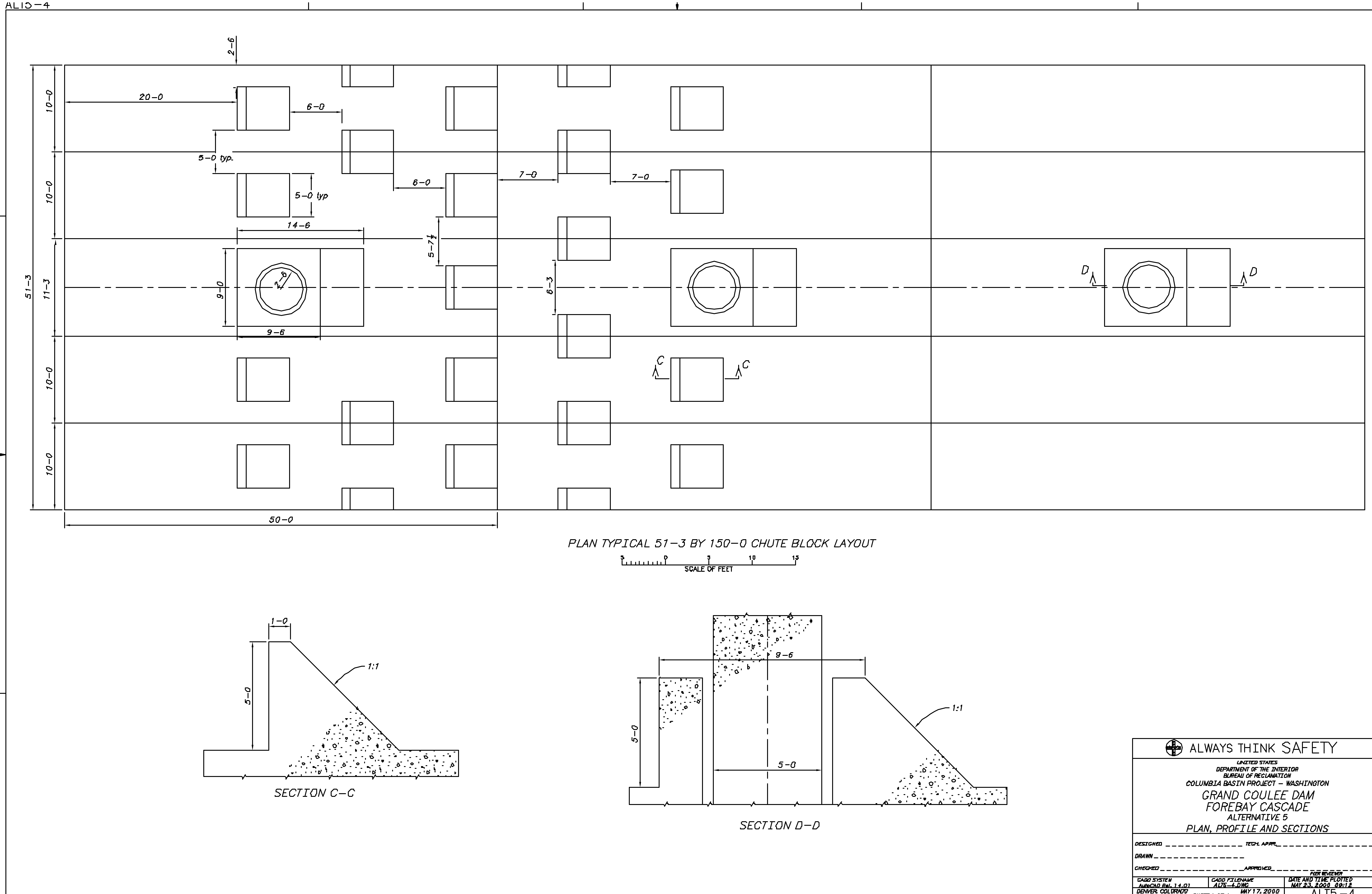
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT – WASHINGTON

GRAND COULEE DAM
FOREBAY CASCADE
ALTERNATIVE 5
PLAN, PROFILE AND SECTIONS

DESIGNED	TECH. APPR.	
DRAWN		
CHECKED	APPROVED	PEER REVIEWER
CADD SYSTEM AutoCAD Rel. 14.01	CADD FILENAME ALT5-3.DWG	DATE AND TIME PLOTTED OCTOBER 23, 2000 07:58
DENVER, COLORADO	SHEET 3 OF 4 MAY 18, 2000	ALT5-3

Drawing of forebay pipe with cascade alternative. Sheet 3 – FIGURE 39



Drawing of forebay pipe with cascade alternative, Sheet 4 – FIGURE 4.0

Figure 41. **-** Grand Coulee Dam forebay cascade model with the conceptual design for initial model study.

Figure 42. **-** Baffled apron over the 2:1 sloping cascade section with the baffled, adjustable horizontal basin and apron. The flow shown is the design flow with reservoir El. 1290 and tailwater El. 965.

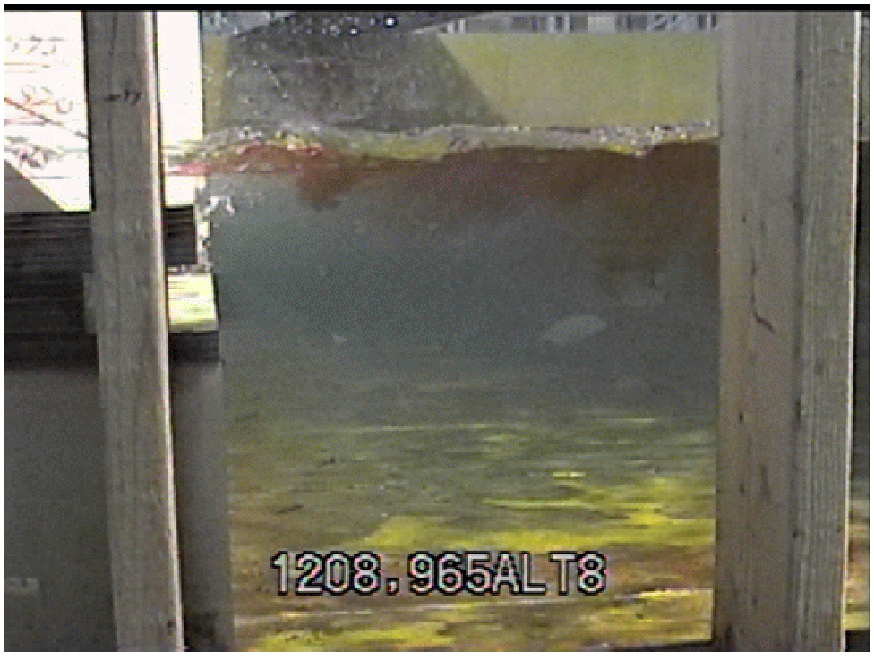


Figure 43. **-** Surface dye injection for Res. El. 1208 and tailwater El 965.

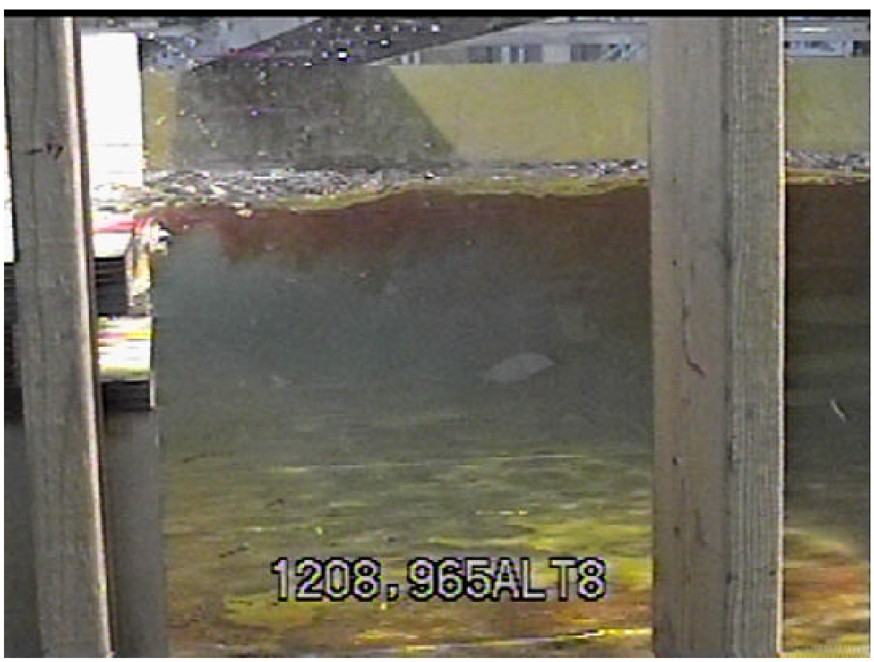


Figure 44. **-** Bottom dye injection for reservoir El. 1208 and tailwater El 965.

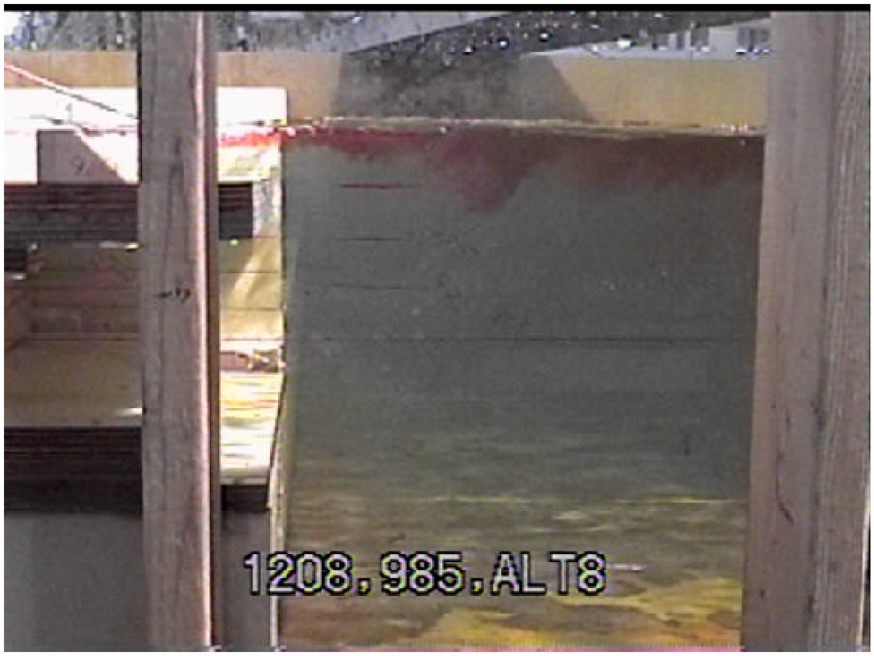


Figure 45. **-** Surface dye injection for reservoir El. 1208 and tailwater El 985.

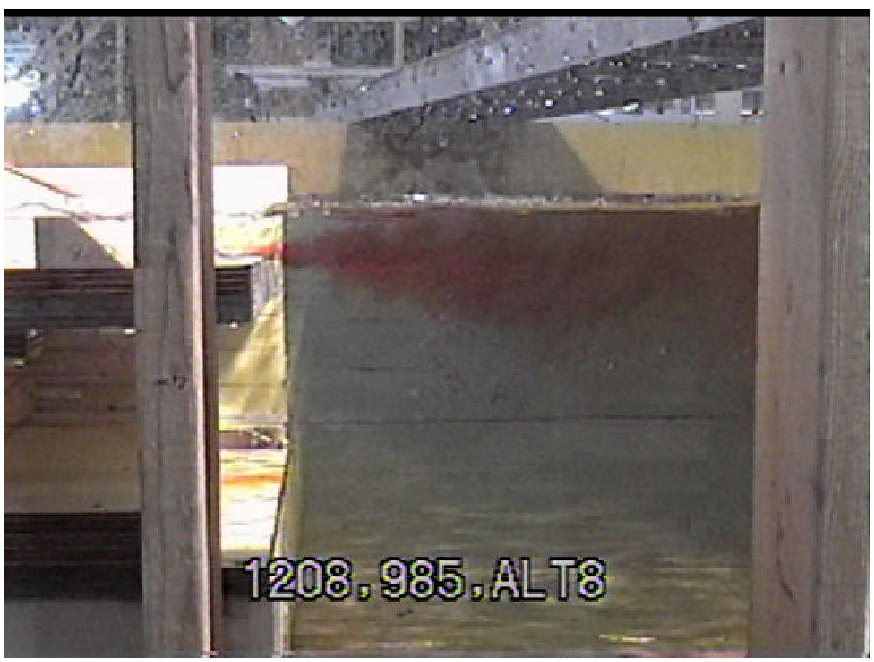


Figure 46. **-** Bottom dye injection for reservoir El. 1208 and tailwater EL. 985.

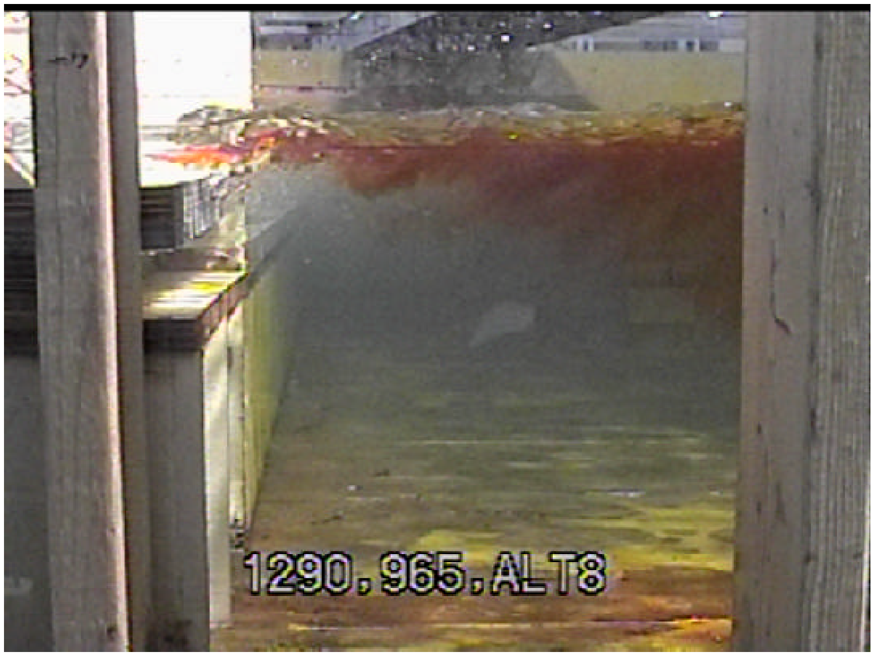


Figure 47. **-** Surface dye injection at reservoir El. 1290 and tailwater El. 965.

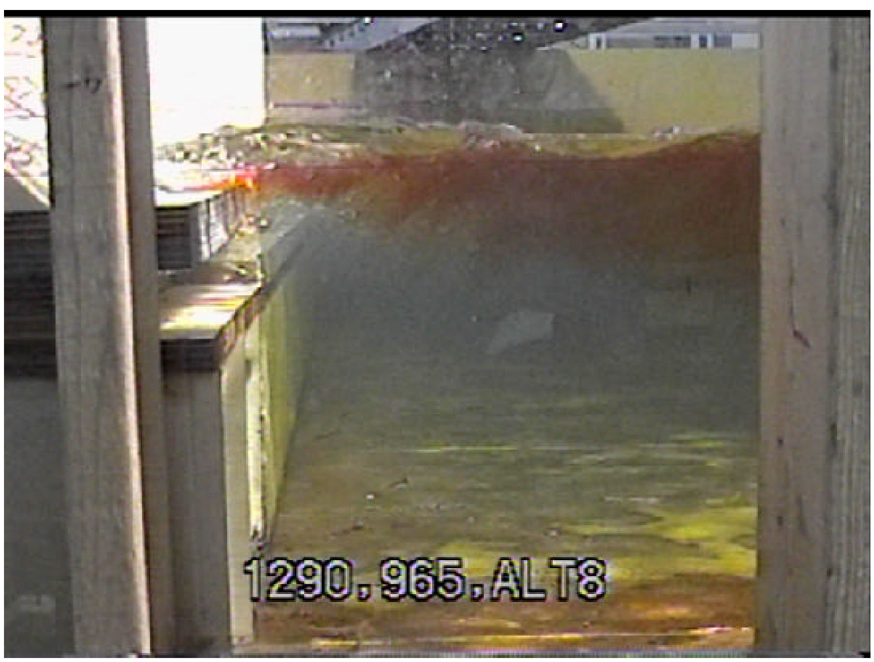


Figure 48. **-** Bottom dye injection for reservoir El. 1290 and tailwater EL. 965.

Figure 49. **-** Surface dye injection for reservoir El. 1290 and tailwater El. 985.

Figure 50. **-** Bottom dye injection for reservoir El. 1290 and tailwater El. 985.

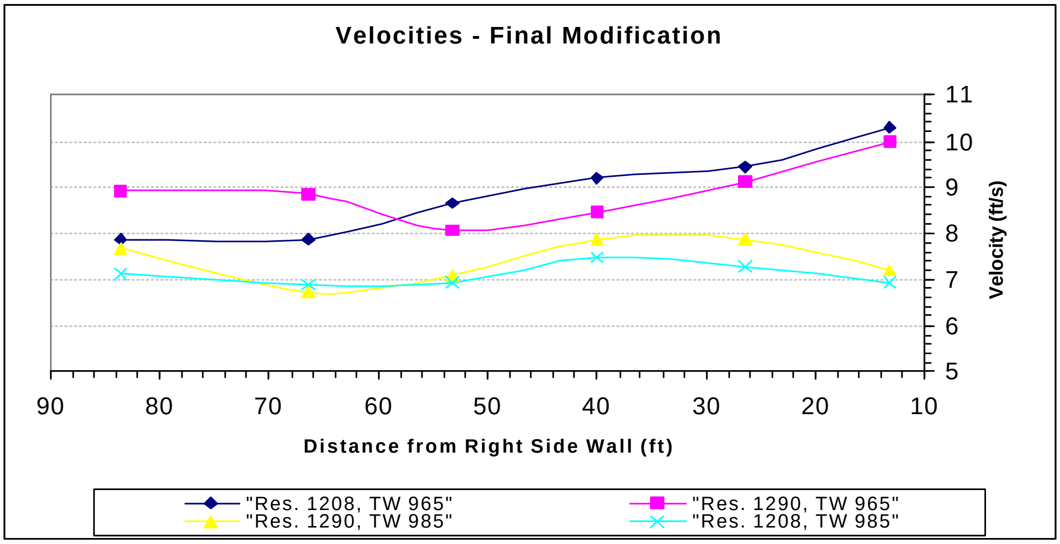


Figure 51. - Velocities measured across the final cascade section for the design flow and reservoir Els. 1208 and 1290 with tailwater Els. 965 to 985.

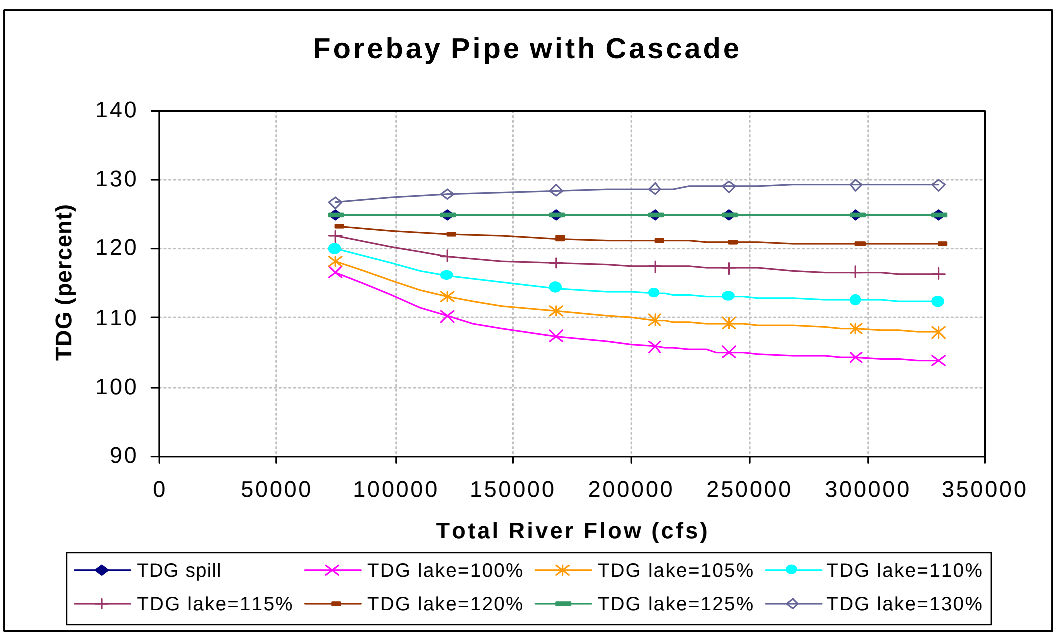


Figure 52. - TDG levels for the forebay pipe with cascade alternative with the design flow rate of 50,000 ft^3/s. The TDG level produced by the spill is constant due to the adjustable stilling basin. The total river flow TDG levels vary with the reservoir TDG levels.

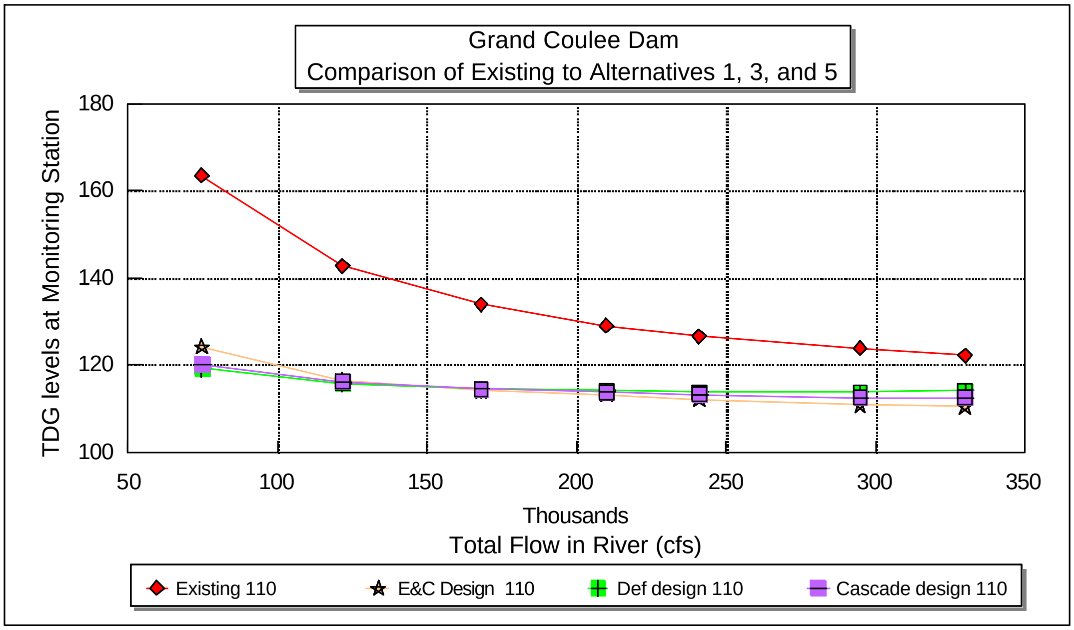


Figure 53. - Comparison of the existing outlet works and alternatives for reservoir TDG level of 110 percent and various total river flows.

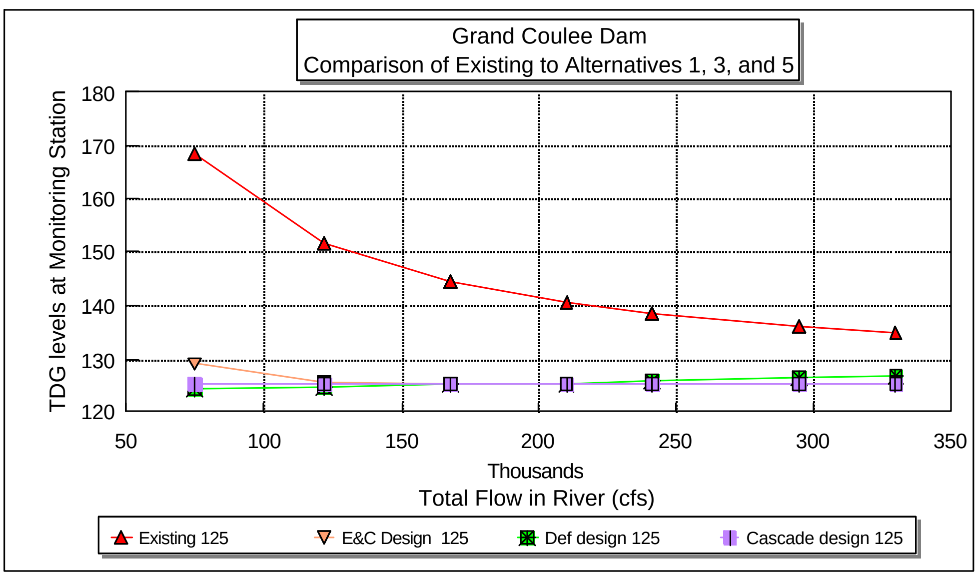


Figure 54. - Comparison of the existing outlet works and alternatives for reservoir TDG level at 125 percent and various total river flows.

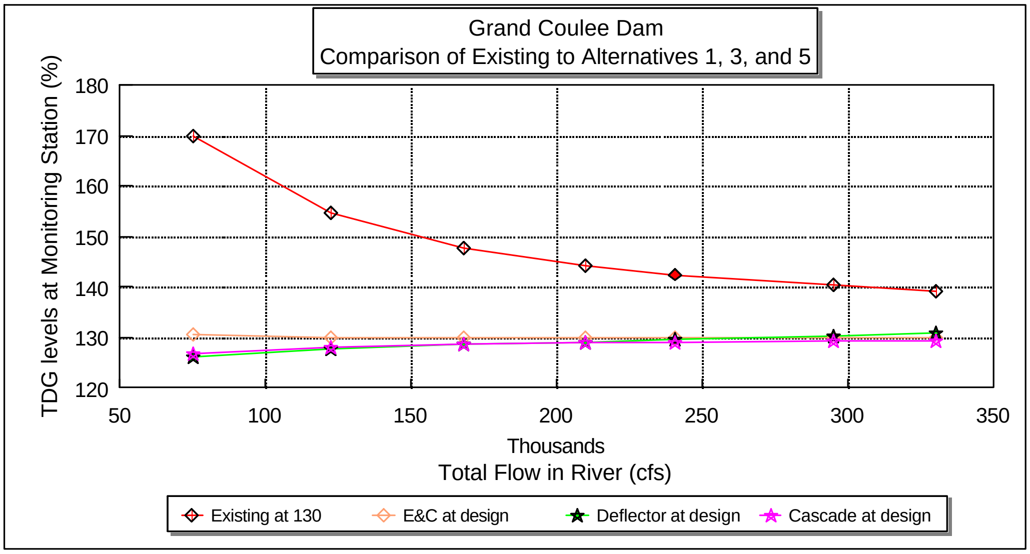


Figure 55. - Comparison of the existing outlet works and alternatives for reservoir TDG level of 130 percent and various total river flows.

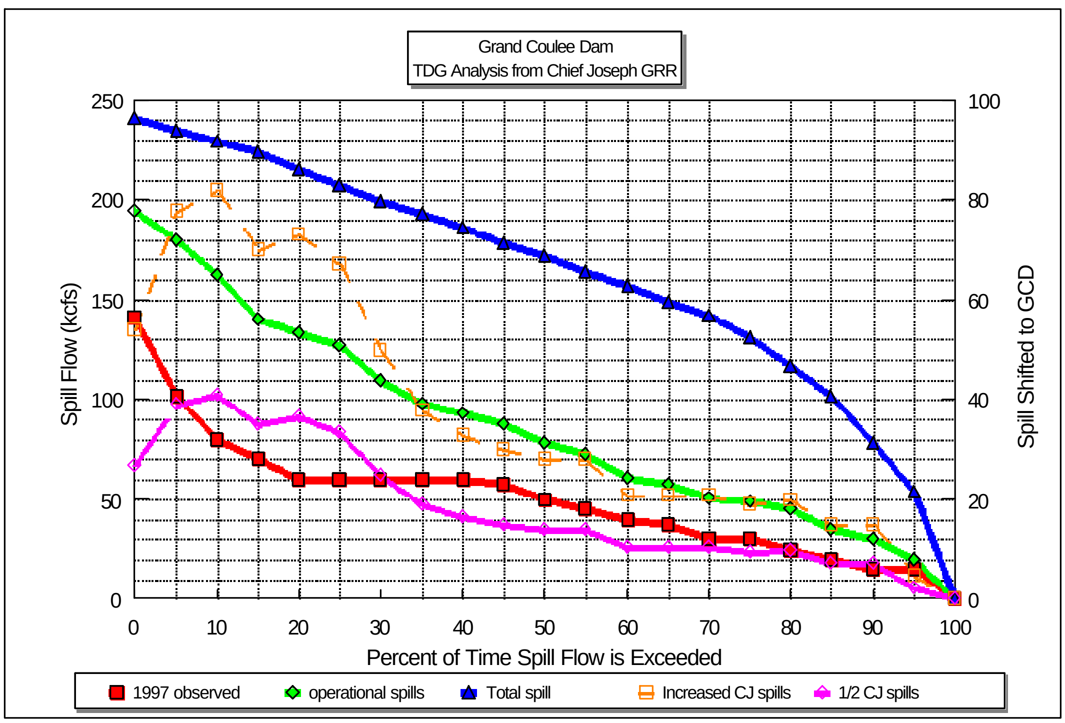


Figure 56. - Data and computation of the flow transferred to Grand Coulee from Chief Joseph from the COE Chief Joseph Gas Abatement General Reevaluation Report, April 2000.

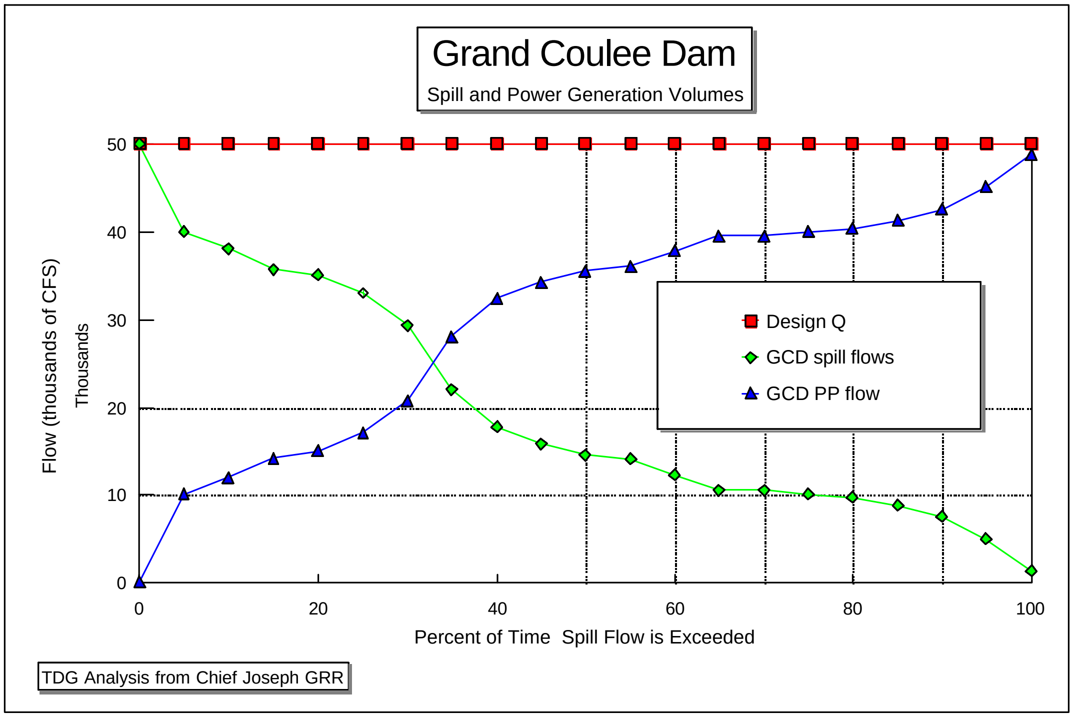


Figure 57. - Spill exceedence for flows at Grand Coulee Dam. Design flow is constant at 50,000 ft^3/s. The transferred spill flow that has been subtracted from the design flow is shown as GCD spill flows. The powerplant flow is the additional power that will be generated at Grand Coulee Dam and is the difference between the 50,000 ft^3/s design flow and the spill flow.

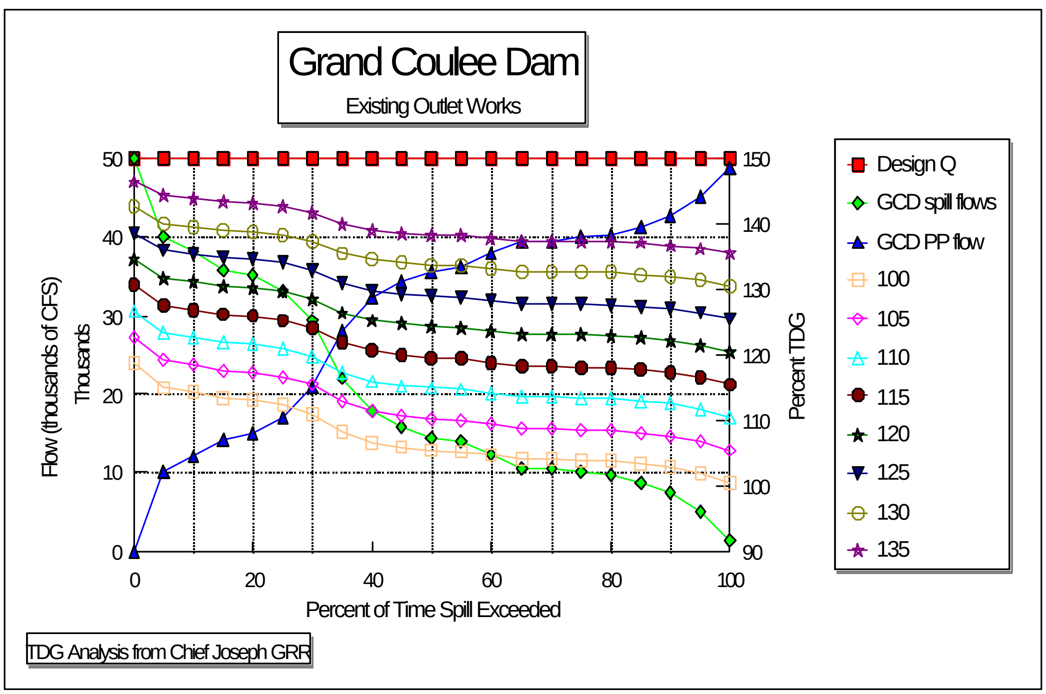


Figure 58. Spill exceedence curves for spills, power and TDG for the existing outlet works with joint operation. The curves were developed using the TDG value for the existing outlet works for the 7Q10 event.

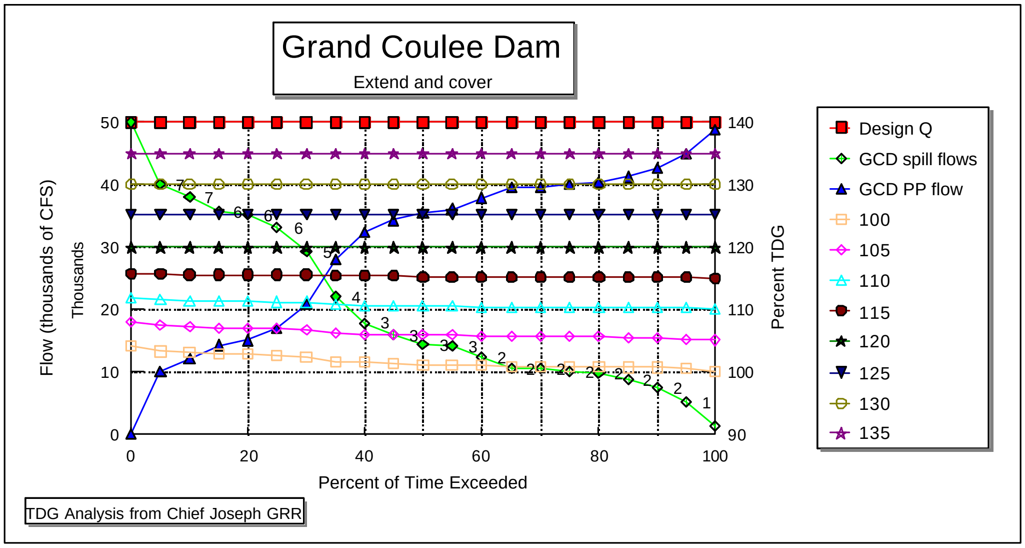


Figure 59. - Spill exceedence for the extend and cover alternative with joint operation. The curves were developed using the TDG value for the extend and cover alternative at the 7Q10 event. The number of pairs of outlets that would be modified based upon the percent exceedence of spill are shown as labels along the GCD spill flow curve. Eight outlet pairs are needed to pass the design flow without joint operation.

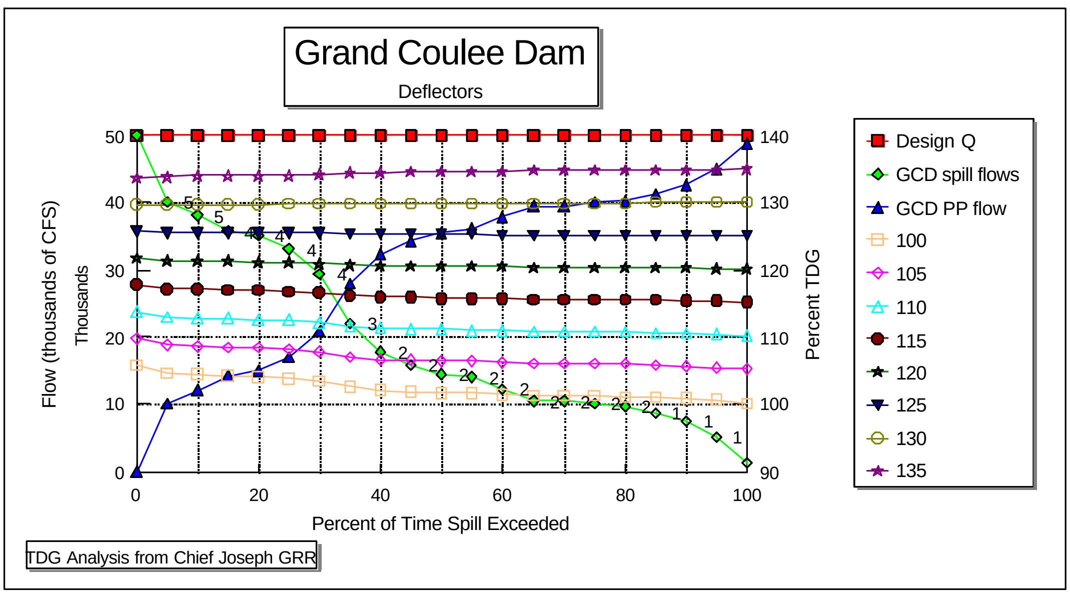


Figure 60. - Spill exceedence for the deflector alternative at Grand Coulee Dam assuming joint operation. The number of blocks that would be modified to pass the flows are shown on the GCD spill flows line. TDG for the deflector calculation was that of the 7Q10 event. Six blocks are needed to pass the design flow without joint operation.

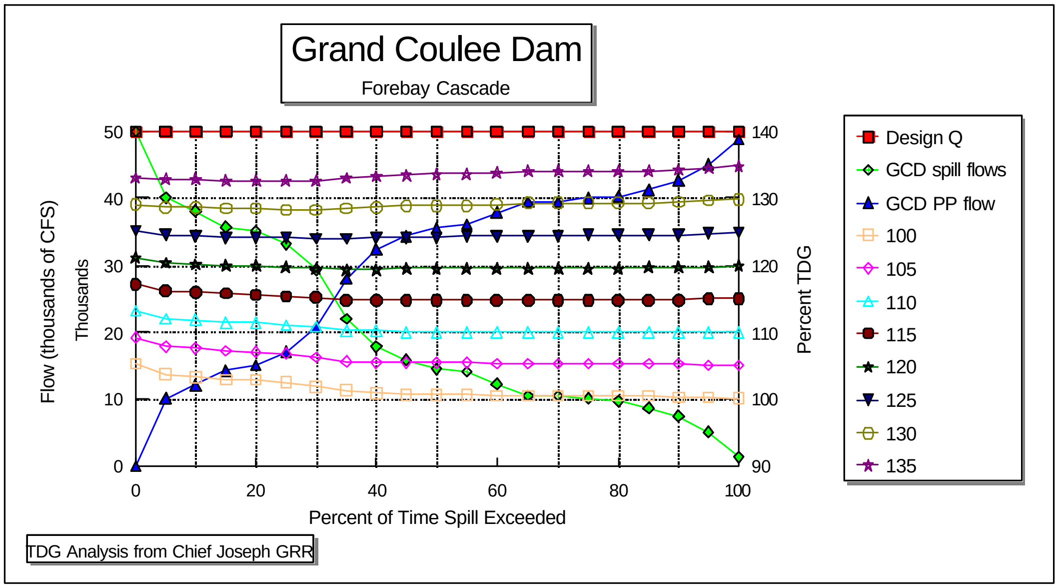


Figure 61. - Spill exceedence for the forebay pipe with cascade alternative at Grand Coulee Dam assuming joint operation. The spill line indicates the amount of flow that could be transferred to Grand Coulee to reduce the unit discharge over the cascade, thus improving the TDG levels as the unit discharge decreases. No modification would be made to the structure.

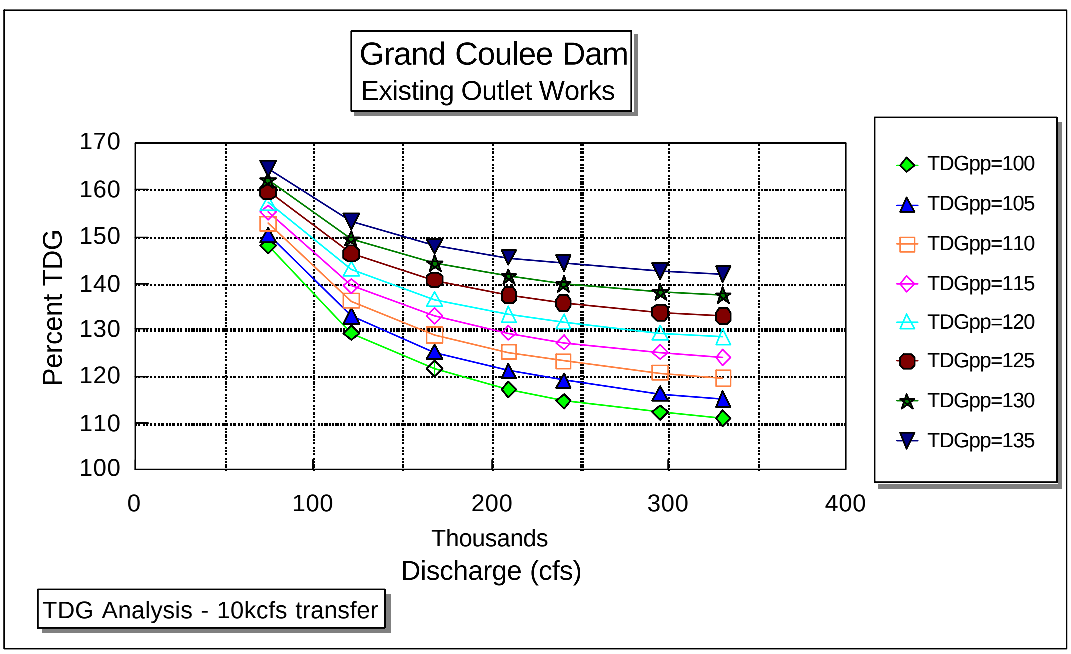


Figure 62. - TDG analysis for the existing outlet works with a flow transfer of 10,000 ft^3/s under joint operation. This would be a total spill discharge of 40,000 ft^3/s.

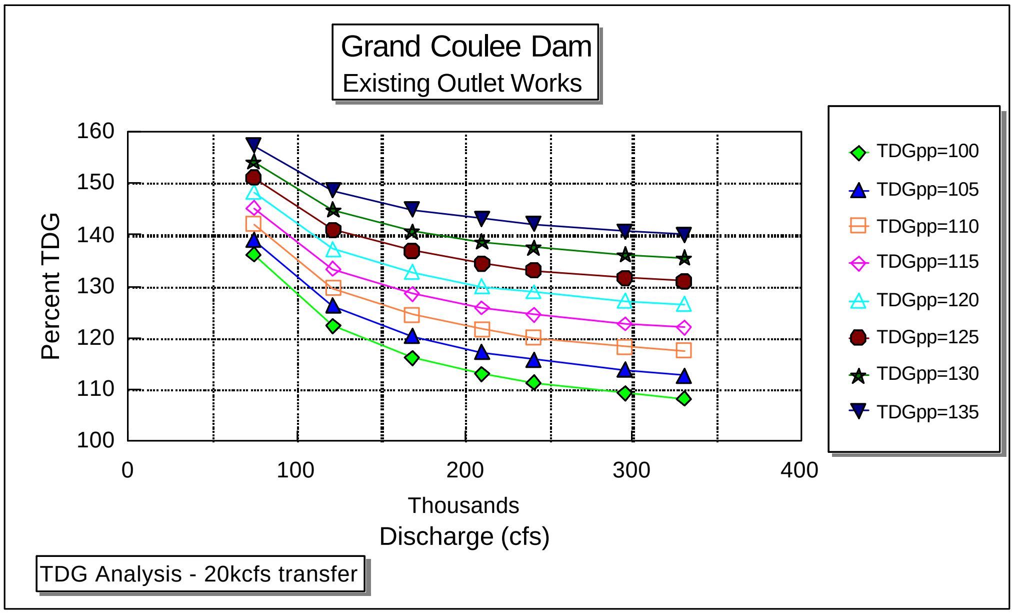


Figure 63. - TDG analysis for the existing outlet works with a flow transfer of 20,000 ft^3/s under joint operation. This would be a total spill discharge of 30,000 ft^3/s.

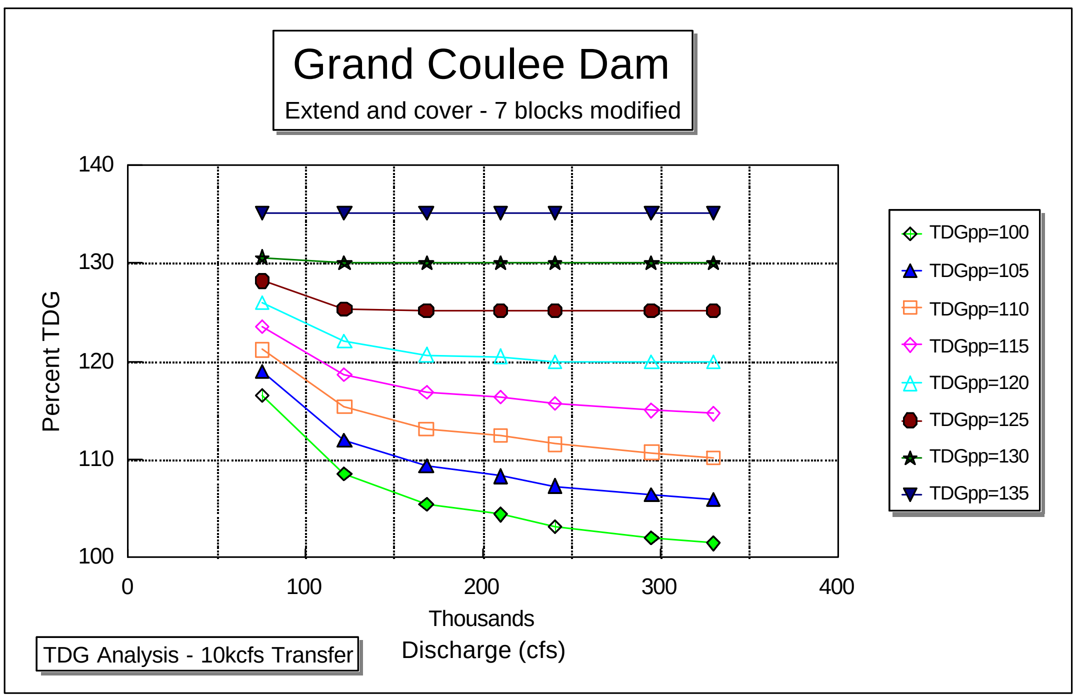


Figure 64. - TDG analysis for the extend and cover alternative with 10,000 ft^3/s transferred under joint operation with Chief Joseph Dam. Seven blocks or pairs of outlets would be modified to discharge a total of 40,000 ft^3/s.

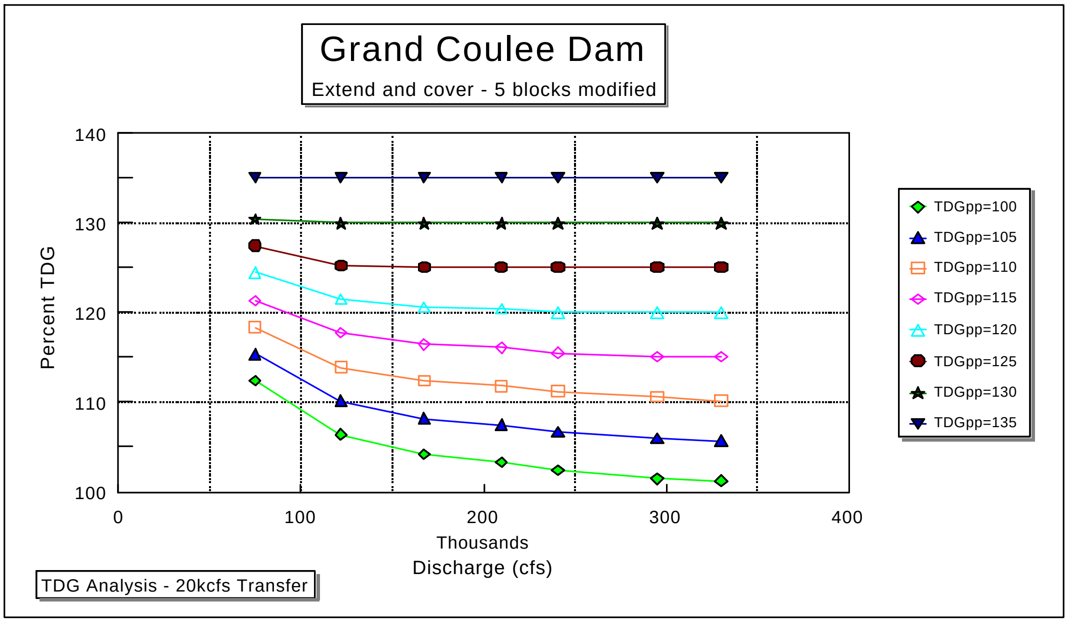


Figure 65. - TDG analysis for the extend and cover alternative with 20,000 ft^3/s transferred under joint operation with Chief Joseph Dam. Five blocks or pairs of outlets would be modified to discharge a total of 30,000 ft^3/s.

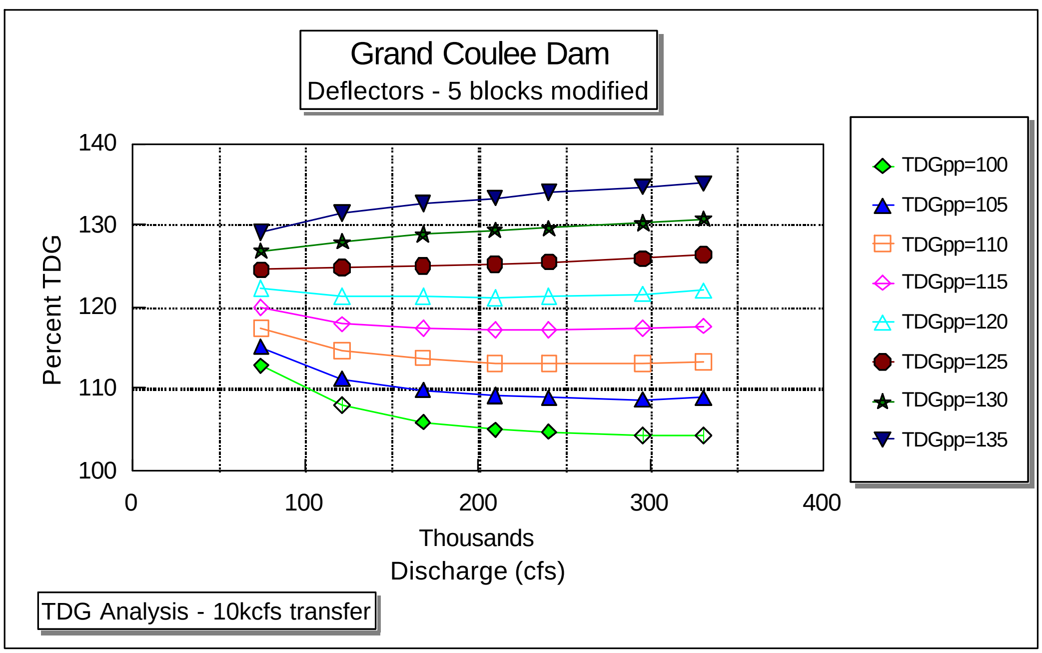


Figure 66. - TDG analysis for the deflector alternative with 10,000 ft^3/s transferred under joint operation with Chief Joseph Dam. Five blocks or pairs of outlets would be modified for a discharge total of 40,000 ft^3/s.

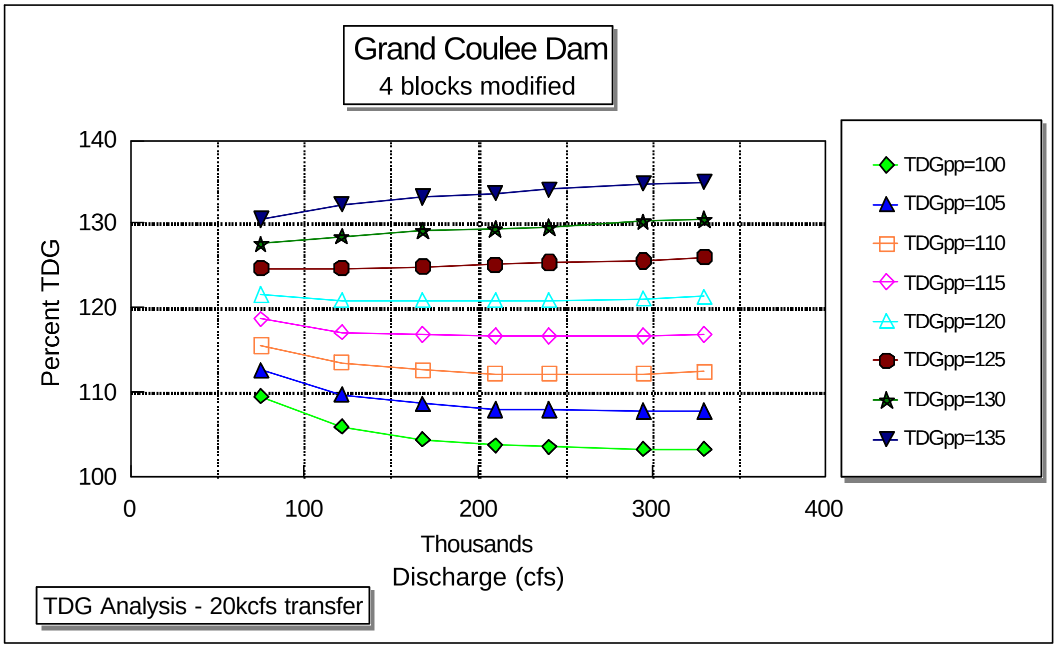


Figure 67. - TDG analysis for the deflector alternative with 20,000 ft^3/s transferred under joint operation with Chief Joseph Dam. Four blocks or pairs of outlets would be modified for a discharge total of 30,000 ft^3/s.

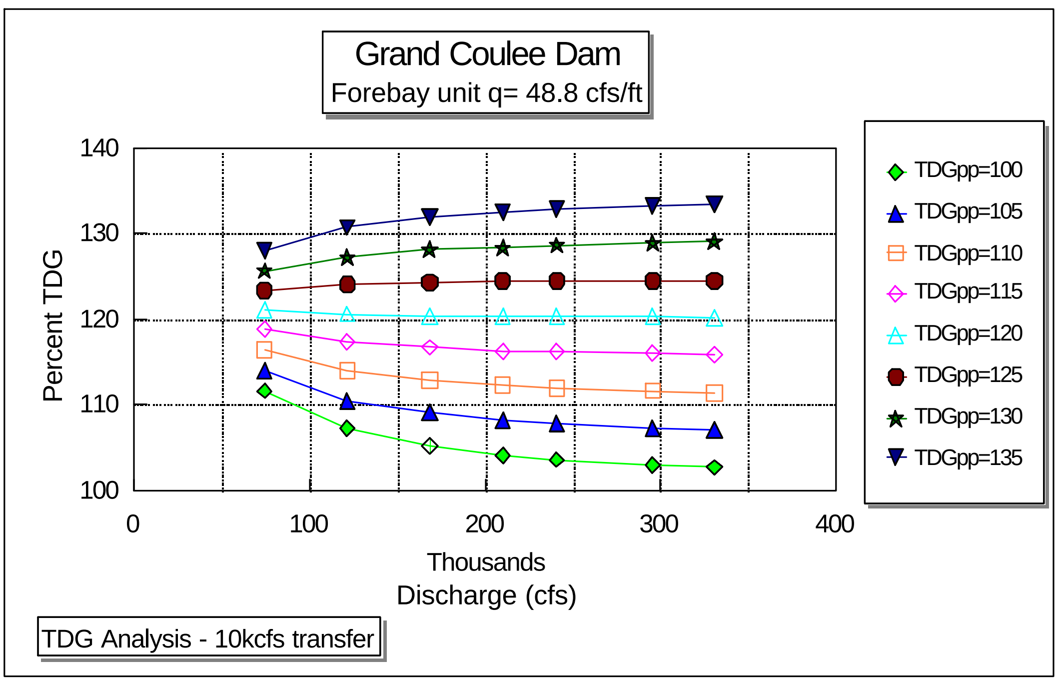


Figure 68. - TDG analysis for the forebay pipe with cascade alternative with 10,000 ft^3/s transferred under joint operation with Chief Joseph Dam. The width of the alternative remains the same, however, the TDG level of the spill is reduced.

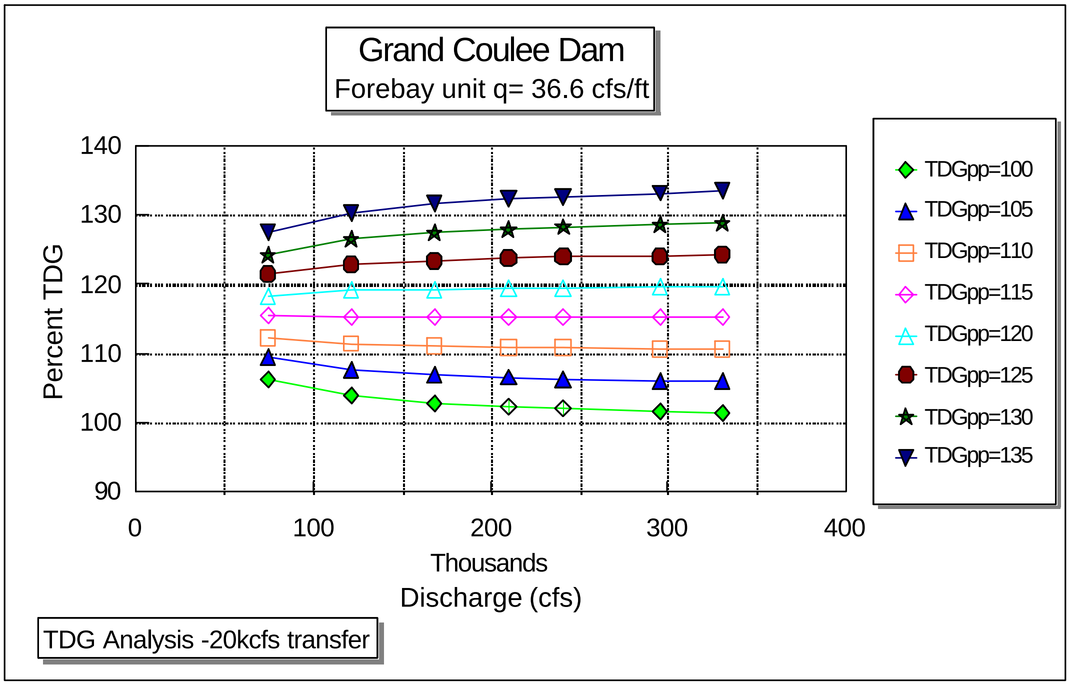


Figure 69. - TDG analysis for the forebay pipe with cascade alternative with 20,000 ft^3/s transferred under joint operation with Chief Joseph Dam. The width of the alternative remains the same, however, the TDG level of the spill is reduced.

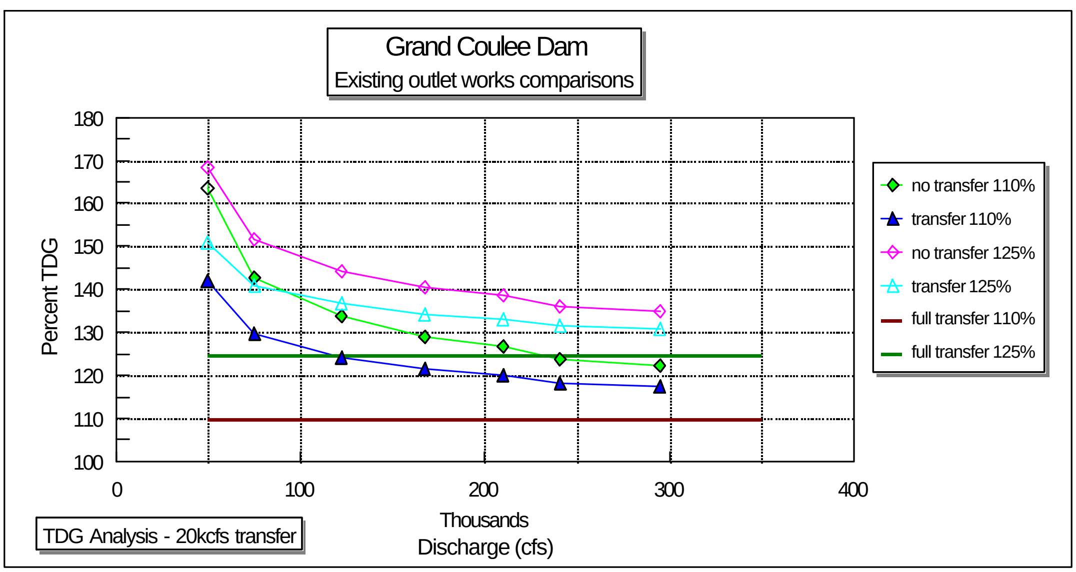


Figure 70. - Comparisons of the TDG levels expected for the existing outlet works flow conditions with and without spill transfer of 20,000 ft^3/s at 110 and 125 percent reservoir TDG levels.

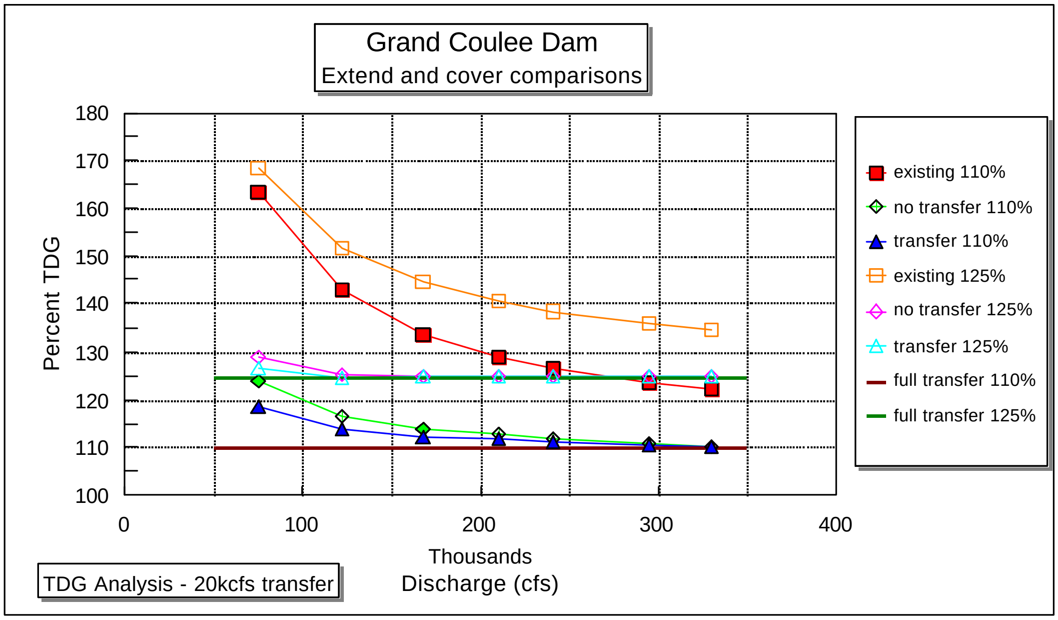


Figure 71. **-** Comparisons of the TDG levels expected for the existing spill condition, the full extend and cover alternative, and the alternative with spill transfer at 110 and 125 percent reservoir levels.

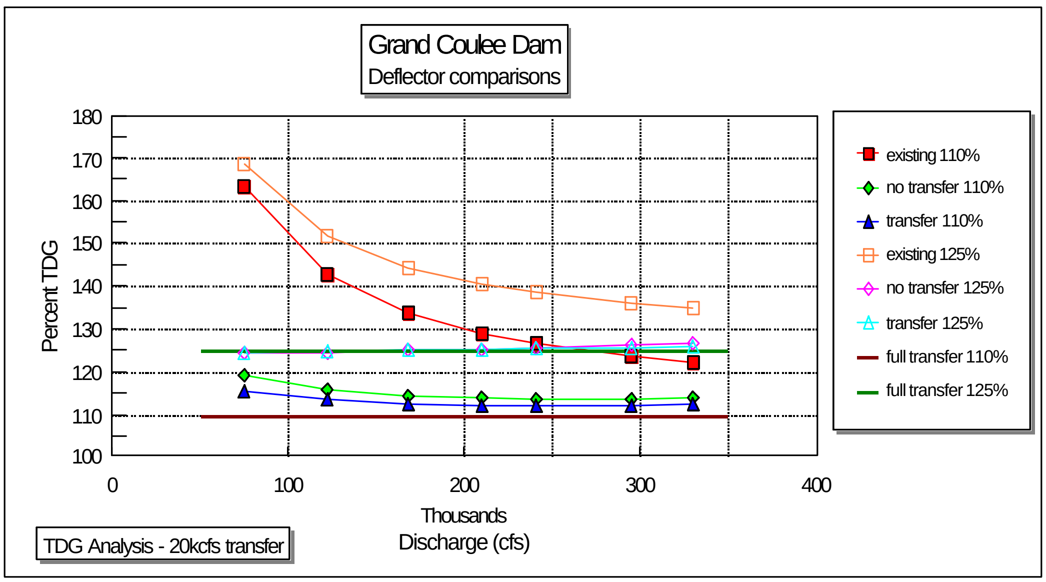


Figure 72. **-** Comparisons of the TDG levels expected for the existing spill condition, the full deflector alternative, and the alternative with spill transfer at 110 and 125 percent reservoir levels.

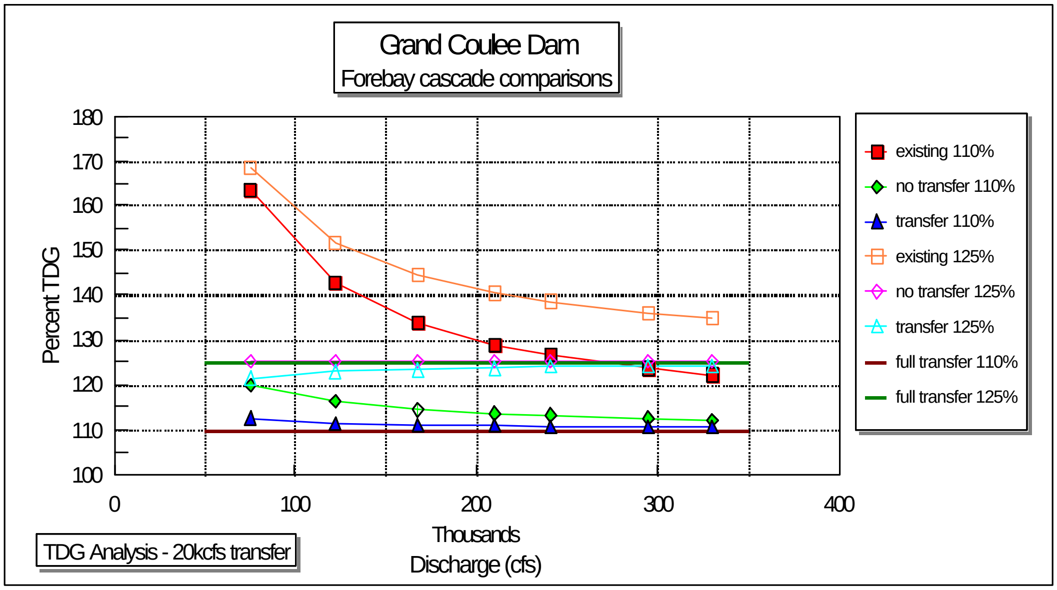


Figure 73. **-** Comparisons of the TDG levels expected for the existing spill condition, the full forebay pipe with cascade alternative, and the alternative with spill transfer at 110 and 125 percent reservoir levels.

APPENDIX 1

Estimate sheets for the Cover and Extend Mid-Level Outlet Works

CODE:D-8130		ESTIMATE WORKSHEET				Page 1 of 10
FEATURE:		27-JUL-00	PROJECT:			
	Grand Coolee Dam		**Columbia Basin Project**			
	Total Disolved Gas Study					
	Extend Outlet Works		DIVISION:	Civil Engineering		
		(FOR 1 BLOCK)	UNIT:	D-8170 - Estimates		
			FILE:			

PAY ITEM	DESCRIPTION	CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
	Mobilization (at 5% of other items)		1	ls	$500,000.00	$500,000
						$0
	Furnish Cofferdam		1	ea	$500,000.00	$500,000
						$0
	Install & Move Cofferdam (h=70'; w= 50')		1	ea	$175,000.00	$175,000
						$0
	Excavation - concrete		5,150	cy	$1,100.00	$5,665,000
	Sawcut (3" deep)		590	lf	$14.00	$8,260
	Drilling for #11 anchor bars (2" dia holes)		430	lf	$28.00	$12,040
	Concrete:					$0
	Reinforced - face of dam		3,290	cy	$375.00	$1,233,750
	Reinforced - backfill		360		$325.00	$117,000
						$0
	Furnishing & Handling Cementitious Material		990	tons	$118.00	$116,820
						$0
	Furnish & Install Reinforcement Bars		358,000	lbs	$0.70	$250,600
	Furnish and Install 8'-6" dia steel pipe		286,000	lbs	$3.25	$929,500
						$0
	Furnish and Install 8'-6" diffuser section and support		36,000	lbs	$3.75	$135,000
						$0
	Furnish and Install 1'-0" dia. vent pipe and valve		1,430	lbs	$4.25	$6,078
						$0
	Unwatering (7-blocks)		1	ea	$10,000.00	$10,000
						$0
	Dewatering (7-blocks, 5-months/block)		5	mo	$30,000.00	$150,000
	Mobilize Barges (1 large, 1 transit)		1	ls	$100,000.00	$100,000
						$0
	Mobilize Cranes (1 shore; 1 barge)		1	ls	$40,000.00	$40,000
						$0
	Barge Rental (6-months per block)		6	mo	$35,000.00	$210,000
						$0
	Crane Rate (operated), large barge crane & smaller shore crane		6	mo	$85,000.00	$510,000
	Subtotal without mob					$10,169,048
	Subtotal with mob					$10,669,048
	Unlisted items @ 10%					$830,952
	Contract Cost					**$11,500,000**
	Contingencies @ 20%					$2,500,000
	FIELD COST					**$14,000,000**

	QUANTITIES			PRICES		
BY	B Cohen		BY	Bill Holbert	CHECKED	
					N. Hyndman 07/27/00	
DATE PREPARED		APPROVED	DATE	07/27/00	PRICE LEVEL	Feasibility
		E Hall				

CODE:D-8130	ESTIMATE WORKSHEET					Page 2 of 10
FEATURE:		PROJECT:				
	Grand Coolee Dam		Columbia Basin Project			
	Total Disolved Gas Study					
	Extend Outlet Works	DIVISION:		Civil Engineering		
	(FOR 2 BLOCKS)	UNIT:		D-8170 - Estimates		
		FILE:				

PAY ITEM	DESCRIPTION	CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
	Mobilization (at 5% of other items)		1	ls	$1,000,000.00	$1,000,000
	Furnish Cofferdam		1	ea	$500,000.00	$500,000
	Install & Move Cofferdam (h=70'; w= 50')		2	ea	$175,000.00	$350,000
	Excavation - concrete		11,000	cy	$1,050.00	$11,550,000
	Sawcut (3" deep)		1,200	lf	$13.00	$15,600
	Drilling for #11 anchor bars (2" dia holes)		900	lf	$27.00	$24,300
	Concrete:					
	Reinforced - face of dam		7,000	cy	$350.00	$2,450,000
	Reinforced - backfill		720		$300.00	$216,000
	Furnishing & Handling Cementitious Material		2,000	tons	$116.00	$232,000
	Furnish & Install Reinforcement Bars		720,000	lbs	$0.70	$504,000
	Furnish and Install 8'-6" dia steel pipe		580,000	lbs	$3.00	$1,740,000
	Furnish and Install 8'-6" diffuser section and support		72,000	lbs	$3.50	$252,000
	Furnish and Install 1'-0" dia. vent pipe and valve		2,900	lbs	$4.00	$11,600
	Unwatering (7-blocks)		2	ea	$10,000.00	$20,000
	Dewatering (7-blocks, 5-months/block)		10	mo	$30,000.00	$300,000
	Mobilize Barges (1 large, 1 transit)		1	ls	$100,000.00	$100,000
	Mobilize Cranes (1 shore; 1 barge)		1	ls	$40,000.00	$40,000
	Barge Rental (6-months per block)		12	mo	$35,000.00	$420,000
	Crane Rate (operated), large barge crane & smaller shore crane		12	mo	$85,000.00	$1,020,000
	Subtotal without mob					$19,745,500
	Subtotal with mob					$20,745,500
	Unlisted items @ 10%					$2,254,500
	Contract Cost					**$23,000,000**
	Contingencies @ 20%					$4,500,000
	FIELD COST					**$27,500,000**

	QUANTITIES		PRICES		
BY		BY	Bill Holbert	CHECKED	
DATE PREPARED	APPROVED	DATE		PRICE LEVEL	Feasibility

CODE:D-8130	ESTIMATE WORKSHEET					Page 3 of 10
FEATURE:		PROJECT:				
	Grand Coolee Dam		Columbia Basin Project			
	Total Disolved Gas Study					
	Extend Outlet Works	DIVISION:		Civil Engineering		
	(FOR 3 BLOCKS)	UNIT:		D-8170 - Estimates		
		FILE:				
PAY ITEM	DESCRIPTION	CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
	Mobilization (at 5% of other items)		1	ls	$1,400,000.00	$1,400,000
	Furnish Cofferdam		2	ea	$500,000.00	$1,000,000
	Install & Move Cofferdam (h=70'; w= 50')		3	ea	$175,000.00	$525,000
	Excavation - concrete		16,000	cy	$1,000.00	$16,000,000
	Sawcut (3" deep)		1,800	lf	$12.00	$21,600
	Drilling for #11 anchor bars (2" dia holes)		1,300	lf	$26.00	$33,800
	Concrete:					
	Reinforced - face of dam		10,000	cy	$325.00	$3,250,000
	Reinforced - backfill		1,080	cy	$275.00	$297,000
	Furnishing & Handling Cementitious Material		3,000	tons	$114.00	$342,000
	Furnish & Install Reinforcement Bars		1,080,000	lbs	$0.65	$702,000
	Furnish and Install 8'-6" dia steel pipe		860,000	lbs	$2.75	$2,365,000
	Furnish and Install 8'-6" diffuser section and support		108,000	lbs	$3.25	$351,000
	Furnish and Install 1'-0" dia. vent pipe and valve		4,300	lbs	$3.75	$16,125
	Unwatering (7-blocks)		3	ea	$10,000.00	$30,000
	Dewatering (7-blocks, 5-months/block)		15	mo	$30,000.00	$450,000
	Mobilize Barges (1 large, 1 transit)		1	ls	$100,000.00	$100,000
	Mobilize Cranes (1 shore; 1 barge)		1	ls	$40,000.00	$40,000
	Barge Rental (6-months per block)		18	mo	$35,000.00	$630,000
	Crane Rate (operated), large barge crane & smaller shore crane		18	mo	$85,000.00	$1,530,000
	Subtotal without mob					$27,683,525
	Subtotal with mob					$29,083,525
	Unlisted items @ 10%					$2,916,475
	Contract Cost					**$32,000,000**
	Contingencies @ 20%					$6,000,000
	FIELD COST					**$38,000,000**
	QUANTITIES		PRICES			
BY		BY	Bill Holbert	CHECKED		
DATE PREPARED	APPROVED	DATE		PRICE LEVEL		Feasibility

FEATURE:			PROJECT:				
	Grand Coolee Dam			**Columbia Basin Project**			
	Total Disolved Gas Study						
	Extend Outlet Works		DIVISION:		Civil Engineering		
		(FOR 4 BLOCKS)	UNIT:		D-8170 - Estimates		
			FILE:				
PAY ITEM	DESCRIPTION		CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
	Mobilization (at 5% of other items)			1	ls	$1,700,000.00	$1,700,000
	Furnish Cofferdam			2	ea	$500,000.00	$1,000,000
	Install & Move Cofferdam (h=70'; w= 50')			4	ea	$175,000.00	$700,000
	Excavation - concrete			21,000	cy	$950.00	$19,950,000
	Sawcut (3" deep)			2,400	lf	$11.00	$26,400
	Drilling for #11 anchor bars (2" dia holes)			1,800	lf	$25.00	$45,000
	Concrete:						
	Reinforced - face of dam			14,000	cy	$300.00	$4,200,000
	Reinforced - backfill			1,430		$250.00	$357,500
	Furnishing & Handling Cementitious Material			4,000	tons	$112.00	$448,000
	Furnish & Install Reinforcement Bars			1,430,000	lbs	$0.65	$929,500
	Furnish and Install 8'-6" dia steel pipe			1,150,000	lbs	$2.50	$2,875,000
	Furnish and Install 8'-6" diffuser section and support			143,000	lbs	$3.00	$429,000
	Furnish and Install 1'-0" dia. vent pipe and valve			5,800	lbs	$3.50	$20,300
	Unwatering (7-blocks)			4	ea	$10,000.00	$40,000
	Dewatering (7-blocks, 5-months/block)			20	mo	$30,000.00	$600,000
	Mobilize Barges (1 large, 1 transit)			1	ls	$100,000.00	$100,000
	Mobilize Cranes (1 shore; 1 barge)			1	ls	$40,000.00	$40,000
	Barge Rental (6-months per block)			24	mo	$35,000.00	$840,000
	Crane Rate (operated), large barge crane & smaller shore crane			24	mo	$85,000.00	$2,040,000
		Subtotal without mob					$34,640,700
		Subtotal with mob					$36,340,700
	Unlisted items @ 10%						$3,659,300
	Contract Cost						**$40,000,000**
	Contingencies @ 20%						$8,000,000
	FIELD COST						**$48,000,000**
	QUANTITIES			PRICES			
BY			BY	Bill Holbert	CHECKED		
DATE PREPARED		APPROVED	DATE		PRICE LEVEL		Feasibility

FEATURE:		PROJECT:				
	Grand Coolee Dam		**Columbia Basin Project**			
	Total Disolved Gas Study					
	Extend Outlet Works	DIVISION:		Civil Engineering		
	(FOR 5 BLOCKS)	UNIT:		D-8170 - Estimates		
		FILE:				
PAY ITEM	DESCRIPTION	CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
	Mobilization (at 5% of other items)		1	ls	$2,000,000.00	$2,000,000
	Furnish Cofferdam		2	ea	$500,000.00	$1,000,000
	Install & Move Cofferdam (h=70'; w= 50')		5	ea	$175,000.00	$875,000
	Excavation - concrete		26,000	cy	$900.00	$23,400,000
	Sawcut (3" deep)		3,000	lf	$10.00	$30,000
	Drilling for #11 anchor bars (2" dia holes)		2,200	lf	$24.00	$52,800
	Concrete:					
	Reinforced - face of dam		17,000	cy	$275.00	$4,675,000
	Reinforced - backfill		1,790		$225.00	$402,750
	Furnishing & Handling Cementitious Material		5,000	tons	$110.00	$550,000
	Furnish & Install Reinforcement Bars		1,790,000	lbs	$0.60	$1,074,000
	Furnish and Install 8'-6" dia steel pipe		1,430,000	lbs	$2.25	$3,217,500
	Furnish and Install 8'-6" diffuser section and support		179,000	lbs	$2.75	$492,250
	Furnish and Install 1'-0" dia. vent pipe and valve		7,200	lbs	$3.25	$23,400
	Unwatering (7-blocks)		5	ea	$10,000.00	$50,000
	Dewatering (7-blocks, 5-months/block)		25	mo	$30,000.00	$750,000
	Mobilize Barges (1 large, 1 transit)		1	ls	$100,000.00	$100,000
	Mobilize Cranes (1 shore; 1 barge)		1	ls	$40,000.00	$40,000
	Barge Rental (6-months per block)		30	mo	$35,000.00	$1,050,000
	Crane Rate (operated), large barge crane & smaller shore crane		30	mo	$85,000.00	$2,550,000
	Subtotal without mob					$40,332,700
	Subtotal with mob					$42,332,700
	Unlisted items @ 10%					$4,667,300
	Contract Cost					**$47,000,000**
	Contingencies @ 20%					$9,000,000
	FIELD COST					**$56,000,000**
	QUANTITIES		PRICES			
BY		BY	Bill Holbert	CHECKED		
DATE PREPARED	APPROVED	DATE		PRICE LEVEL		Feasibility

CODE:D-8130	ESTIMATE WORKSHEET					Page 6 of 10
FEATURE:		PROJECT:				
	Grand Coolee Dam		Columbia Basin Project			
	Total Disolved Gas Study					
	Extend Outlet Works	DIVISION:		Civil Engineering		
	(FOR 6 BLOCKS)	UNIT:		D-8170 - Estimates		
		FILE:				
PAY ITEM	DESCRIPTION	CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
	Mobilization (at 5% of other items)		1	ls	$2,300,000.00	$2,300,000
	Furnish Cofferdam		2	ea	$500,000.00	$1,000,000
	Install & Move Cofferdam (h=70'; w= 50')		6	ea	$175,000.00	$1,050,000
	Excavation - concrete		31,000	cy	$850.00	$26,350,000
	Sawcut (3" deep)		3,600	lf	$9.00	$32,400
	Drilling for #11 anchor bars (2" dia holes)		2,600	lf	$23.00	$59,800
	Concrete:					
	Reinforced - face of dam		20,000	cy	$250.00	$5,000,000
	Reinforced - backfill		2,150	cy	$200.00	$430,000
	Furnishing & Handling Cementitious Material		6,000	tons	$108.00	$648,000
	Furnish & Install Reinforcement Bars		2,150,000	lbs	$0.60	$1,290,000
	Furnish and Install 8'-6" dia steel pipe		1,720,000	lbs	$2.00	$3,440,000
	Furnish and Install 8'-6" diffuser section and support		215,000	lbs	$2.50	$537,500
	Furnish and Install 1'-0" dia. vent pipe and valve		8,600	lbs	$3.00	$25,800
	Unwatering (7-blocks)		6	ea	$10,000.00	$60,000
						$0
	Dewatering (7-blocks, 5-months/block)		30	mo	$30,000.00	$900,000
	Mobilize Barges (1 large, 1 transit)		1	ls	$100,000.00	$100,000
	Mobilize Cranes (1 shore; 1 barge)		1	ls	$40,000.00	$40,000
	Barge Rental (6-months per block)		36	mo	$35,000.00	$1,260,000
	Crane Rate (operated), large barge crane & smaller shore crane		36	mo	$85,000.00	$3,060,000
	Subtotal without mob					$45,283,500
	Subtotal with mob					$47,583,500
	Unlisted items @ 10%					$4,416,500
	Contract Cost					**$52,000,000**
	Contingencies @ 20%					$11,000,000
	FIELD COST					**$63,000,000**
	QUANTITIES		PRICES			
BY		BY	Bill Holbert	CHECKED		
DATE PREPARED		APPROVED	DATE	PRICE LEVEL		Feasibility

CODE:D-8130	ESTIMATE WORKSHEET					Page 7 of 10
FEATURE:		PROJECT:				
	Grand Coolee Dam		**Columbia Basin Project**			
	Total Disolved Gas Study					
	Extend Outlet Works	DIVISION:		Civil Engineering		
	(FOR 7 BLOCKS)	UNIT:		D-8170 - Estimates		
		FILE:				
PAY ITEM	DESCRIPTION	CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
	Mobilization (at 5% of other items)		1	ls	$2,500,000.00	$2,500,000
	Furnish Cofferdam		2	ea	$500,000.00	$1,000,000
	Install & Move Cofferdam (h=70'; w= 50')		7	ea	$175,000.00	$1,225,000
	Excavation - concrete		36,000	cy	$800.00	$28,800,000
	Sawcut (3" deep)		4,100	lf	$8.00	$32,800
	Drilling for #11 anchor bars (2" dia holes)		3,000	lf	$22.00	$66,000
	Concrete:					
	Reinforced - face of dam		23,000	cy	$225.00	$5,175,000
	Reinforced - backfill		2,500		$175.00	$437,500
	Furnishing & Handling Cementitious Material		6,900	tons	$106.00	$731,400
	Furnish & Install Reinforcement Bars		2,500,000	lbs	$0.55	$1,375,000
	Furnish and Install 8'-6" dia steel pipe		2,000,000	lbs	$1.75	$3,500,000
	Furnish and Install 8'-6" diffuser section and support		250,000	lbs	$2.25	$562,500
	Furnish and Install 1'-0" dia. vent pipe and valve		10,000	lbs	$2.75	$27,500
	Unwatering (7-blocks)		7	ea	$10,000.00	$70,000
	Dewatering (7-blocks, 5-months/block)		35	mo	$30,000.00	$1,050,000
	Mobilize Barges (1 large, 1 transit)		1	ls	$100,000.00	$100,000
	Mobilize Cranes (1 shore; 1 barge)		1	ls	$40,000.00	$40,000
	Barge Rental (6-months per block)		42	mo	$35,000.00	$1,470,000
	Crane Rate (operated), large barge crane & smaller shore crane		42	mo	$85,000.00	$3,570,000
	Subtotal without mob					$49,232,700
	Subtotal with mob					$51,732,700
	Unlisted items @ 10%					$5,267,300
	Contract Cost					**$57,000,000**
	Contingencies @ 20%					$11,000,000
	FIELD COST					**$68,000,000**
	QUANTITIES		PRICES			
BY		BY	Bill Holbert	CHECKED		
DATE PREPARED		APPROVED	DATE	PRICE LEVEL		Feasibility

CODE:D-8130			ESTIMATE WORKSHEET				Page 8 of 10
FEATURE:			PROJECT:				
	Grand Coolee Dam			**Columbia Basin Project**			
	Total Disolved Gas Study						
	Extend Outlet Works		DIVISION:		Civil Engineering		
		(FOR 8 BLOCKS)	UNIT:		D-8170 - Estimates		
			FILE:				

PAY ITEM	DESCRIPTION			CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
	Mobilization (at 5% of other items)				1	ls	$2,700,000.00	$2,700,000
	Furnish Cofferdam				2	ea	$500,000.00	$1,000,000
								$0
	Install & Move Cofferdam (h=70'; w= 50')				8	ea	$175,000.00	$1,400,000
	Excavation - concrete				42,000	cy	$750.00	$31,500,000
	Sawcut (3" deep)				4,700	lf	$7.00	$32,900
	Drilling for #11 anchor bars (2" dia holes)				3,500	lf	$21.00	$73,500
	Concrete:							
		Reinforced - face of dam			27,000	cy	$200.00	$5,400,000
		Reinforced - backfill			2,860	cy	$150.00	$429,000
	Furnishing & Handling Cementitious Material				7,900	tons	$104.00	$821,600
	Furnish & Install Reinforcement Bars				2,860,000	lbs	$0.55	$1,573,000
	Furnish and Install 8'-6" dia steel pipe				2,290,000	lbs	$1.50	$3,435,000
	Furnish and Install 8'-6" diffuser section and support				286,000	lbs	$2.00	$572,000
	Furnish and Install 1'-0" dia. vent pipe and valve				11,500	lbs	$2.50	$28,750
	Unwatering (7-blocks)				8	ea	$10,000.00	$80,000
	Dewatering (7-blocks, 5-months/block)				40	mo	$30,000.00	$1,200,000
	Mobilize Barges (1 large, 1 transit)				1	ls	$100,000.00	$100,000
	Mobilize Cranes (1 shore; 1 barge)				1	ls	$40,000.00	$40,000
	Barge Rental (6-months per block)				48	mo	$35,000.00	$1,680,000
	Crane Rate (operated), large barge crane & smaller shore crane				48	mo	$85,000.00	$4,080,000
			Subtotal without mob					$53,445,750
			Subtotal with mob					$56,145,750
	Unlisted items @ 10%							$5,854,250
	Contract Cost							**$62,000,000**
	Contingencies @ 20%							$12,000,000
	FIELD COST							**$74,000,000**
		QUANTITIES			PRICES			
BY				BY	Bill Holbert	CHECKED		
DATE PREPARED			APPROVED	DATE		PRICE LEVEL		Feasibility

CODE:D-8130	ESTIMATE WORKSHEET					
FEATURE:		PROJECT:				
	Grand Coulee Dam		**Columbia Basin Project**			
	Total Disolved Gas Study					
	Extend Outlet Works	DIVISION:		Civil Engineering		
	(FOR 9 BLOCKS)	UNIT:		D-8170 - Estimates		
		FILE:				
PAY ITEM	DESCRIPTION	CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
	Mobilization (at 5% of other items)		1	ls	$2,800,000.00	$2,800,000
	Furnish Cofferdam		2	ea	$500,000.00	$1,000,000
	Install & Move Cofferdam (h=70'; w= 50')		9	ea	$175,000.00	$1,575,000
	Excavation - concrete		47,000	cy	$700.00	$32,900,000
	Sawcut (3" deep)		5,300	lf	$6.00	$31,800
	Drilling for #11 anchor bars (2" dia holes)		3,900	lf	$20.00	$78,000
	Concrete:					
	Reinforced - face of dam		30,000	cy	$175.00	$5,250,000
	Reinforced - backfill		3,220	cy	$125.00	$402,500
	Furnishing & Handling Cementitious Material		8,900	tons	$102.00	$907,800
	Furnish & Install Reinforcement Bars		3,220,000	lbs	$0.50	$1,610,000
	Furnish and Install 8'-6" dia steel pipe		2,580,000	lbs	$1.25	$3,225,000
	Furnish and Install 8'-6" diffuser section and support		322,000	lbs	$1.75	$563,500
	Furnish and Install 1'-0" dia. vent pipe and valve		12,900	lbs	$2.25	$29,025
	Unwatering (7-blocks)		9	ea	$10,000.00	$90,000
	Dewatering (7-blocks, 5-months/block)		45	mo	$30,000.00	$1,350,000
	Mobilize Barges (1 large, 1 transit)		1	ls	$100,000.00	$100,000
	Mobilize Cranes (1 shore; 1 barge)		1	ls	$40,000.00	$40,000
	Barge Rental (6-months per block)		54	mo	$35,000.00	$1,890,000
	Crane Rate (operated), large barge crane & smaller shore crane		54	mo	$85,000.00	$4,590,000
	Subtotal without mob					$55,632,625
	Subtotal with mob					$58,432,625
	Unlisted items @ 10%					$5,567,375
	Contract Cost					**$64,000,000**
	Contingencies @ 20%					$13,000,000
	FIELD COST					**$77,000,000**
	QUANTITIES		PRICES			
BY		BY	Bill Holbert	CHECKED		
DATE PREPARED	APPROVED	DATE		PRICE LEVEL		Feasibility

FEATURE: **Grand Coolee Dam** / **Total Disolved Gas Study** / **Extend Outlet Works** **(FOR 10 BLOCKS)**

PROJECT: **Columbia Basin Project**

DIVISION: Civil Engineering

UNIT: D-8170 - Estimates

FILE:

PAY ITEM	DESCRIPTION	CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
	Mobilization (at 5% of other items)		1	ls	$2,900,000.00	$2,900,000
	Furnish Cofferdam		2	ea	$500,000.00	$1,000,000
	Install & Move Cofferdam (h=70'; w= 50')		10	ea	$175,000.00	$1,750,000
	Excavation - concrete		52,000	cy	$650.00	$33,800,000
	Sawcut (3" deep)		5,900	lf	$5.00	$29,500
	Drilling for #11 anchor bars (2" dia holes)		4,300	lf	$19.00	$81,700
	Concrete:					
	Reinforced - face of dam		33,000	cy	$150.00	$4,950,000
	Reinforced - backfill		3,580		$100.00	$358,000
	Furnishing & Handling Cementitious Material		9,900	tons	$100.00	$990,000
	Furnish & Install Reinforcement Bars		3,580,000	lbs	$0.50	$1,790,000
	Furnish and Install 8'-6" dia steel pipe		2,860,000	lbs	$1.00	$2,860,000
	Furnish and Install 8'-6" diffuser section and support		358,000	lbs	$1.50	$537,000
	Furnish and Install 1'-0" dia. vent pipe and valve		14,300	lbs	$2.00	$28,600
	Unwatering (7-blocks)		10	ea	$10,000.00	$100,000
	Dewatering (7-blocks, 5-months/block)		50	mo	$30,000.00	$1,500,000
	Mobilize Barges (1 large, 1 transit)		1	ls	$100,000.00	$100,000
	Mobilize Cranes (1 shore; 1 barge)		1	ls	$40,000.00	$40,000
	Barge Rental (6-months per block)		60	mo	$35,000.00	$2,100,000
	Crane Rate (operated), large barge crane & smaller shore crane		60	mo	$85,000.00	$5,100,000
	Subtotal without mob					$57,114,800
	Subtotal with mob					$60,014,800
	Unlisted items @ 10%					$5,985,200
	Contract Cost					**$66,000,000**
	Contingencies @ 20%					$13,000,000
	FIELD COST					**$79,000,000**

QUANTITIES		PRICES	
BY		BY Bill Holbert	CHECKED
DATE PREPARED	APPROVED	DATE	PRICE LEVEL Feasibility

APPENDIX 2

Estimate sheets for the Deflector Option

CODE:D-8130			ESTIMATE WORKSHEET					Page 1 of 10
FEATURE:			27-Jul-00	PROJECT:				
		Grand Coolee Dam			**Columbia Basin Project**			
		Total Disolved Gas Study						
		Deflectors on face of Spillway		DIVISION:		Civil Engineering		
		68-ft Radius, Price is for 1 Block		UNIT:		D-8170 - Estimates		
				FILE:				
PAY							UNIT	
ITEM	DESCRIPTION			CODE	QUANTITY	UNIT	PRICE	AMOUNT
	Mobilization (at 5% of other items)				1	ls	$110,000.00	$110,000
								$0
	Furnish Cofferdam				1	ea	$500,000.00	$500,000
								$0
	Install & Move Cofferdam (h=70'; w= 50')				1	ea	$175,000.00	$175,000
								$0
	Excavation - concrete				30	cy	$1,500.00	$45,000
								$0
	Surface Preparation of Concrete				3,120	sf	$10.00	$31,200
								$0
	Sawcut (3" deep)				230	lf	$15.00	$3,450
								$0
	Mobilize Barges (1 large, 1 transit)				1	ls	$100,000.00	$100,000
								$0
	Mobilize Cranes (1 shore; 1 barge)				1	ls	$40,000.00	$40,000
								$0
	Barge Rental				3	mo	$35,000.00	$105,000
								$0
	Crane Rate (operated), large barge crane & smaller shore crane				3	mo	$85,000.00	$255,000
								$0
	Drilling for #11 anchor bars (2" dia holes)				6,600	lf	$28.00	$184,800
								$0
	Concrete:							$0
		Reinforced - face of dam			1,210	cy	$350.00	$423,500
								$0
	Furnishing & Handling Cementitious Material				350	tons	$120.00	$42,000
								$0
	Furnish & Install Reinforcement Bars				262,000	lbs	$0.70	$183,400
								$0
	Unwatering				1	ea	$10,000.00	$10,000
								$0
	Dewatering				3	mo	$30,000.00	$90,000
					Subtotal without mobilization			$2,188,350
			Subtotal with mobilization					$2,298,350
	Unlisted Items; 10% (+ or -)							$201,650
	Construction Cost							**$2,500,000**
	Contingencies; 20% (+ or -)							$500,000
	FIELD COST							**$3,000,000**
		QUANTITIES			PRICES			
BY	B Cohen			BY	Bill Holbert	CHECKED		
							Craig A. Grush 7/27/00	
DATE PREPARED			APPROVED	DATE	07/27/00	PRICE LEVEL		Feasibility
			E Hall					

FEATURE:	29-Aug-00	PROJECT:		
Grand Coolee Dam		**Columbia Basin Project**		
Total Disolved Gas Study				
Deflectors on face of Spillway		DIVISION:	Civil Engineering	
68-ft Radius, Price is for 2 Blocks		UNIT:	D-8170 - Estimates	
		FILE:		

PAY ITEM	DESCRIPTION	CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
	Mobilization (at 5% of other items)		1	ls	$180,000.00	$180,000
	Furnish Cofferdam		1	ea	$500,000.00	$500,000
	Install & Move Cofferdam (h=70'; w= 50')		2	ea	$175,000.00	$350,000
	Excavation - concrete		59	cy	$1,500.00	$88,500
	Surface Preparation of Concrete		6,240	sf	$9.00	$56,160
	Sawcut (3" deep)		450	lf	$14.00	$6,300
	Mobilize Barges (1 large, 1 transit)		1	ls	$100,000.00	$100,000
	Mobilize Cranes (1 shore; 1 barge)		1	ls	$40,000.00	$40,000
	Barge Rental		6	mo	$35,000.00	$210,000
	Crane Rate (operated), large barge crane & smaller shore crane		6	mo	$85,000.00	$510,000
	Drilling for #11 anchor bars (2" dia holes)		13,100	lf	$26.00	$340,600
	Concrete:					
	Reinforced - face of dam		2,420	cy	$325.00	$786,500
	Furnishing & Handling Cementitious Material		690	tons	$120.00	$82,800
	Furnish & Install Reinforcement Bars		524,000	lbs	$0.70	$366,800
	Unwatering		2	ea	$10,000.00	$20,000
	Dewatering		6	mo	$30,000.00	$180,000
	Subtotal without mobilization					$3,637,660
	Subtotal with mobilization					$3,817,660
	Unlisted Items; 10% (+ or -)					$382,340
	Construction Cost					**$4,200,000**
	Contingencies; 20% (+ or -)					$800,000
	FIELD COST					**$5,000,000**

	QUANTITIES		PRICES		
BY	B Cohen	BY	Bill Holbert	CHECKED	
				Craig A. Grush 7/27/00	
DATE PREPARED	APPROVED	DATE	07/27/00	PRICE LEVEL	Feasibility
	E Hall				

CODE:D-8130		ESTIMATE WORKSHEET				
FEATURE:		29-Aug-00	PROJECT:			
	Grand Coolee Dam		**Columbia Basin Project**			
	Total Disolved Gas Study					
	Deflectors on face of Spillway		DIVISION:	Civil Engineering		
	68-ft Radius, Price is for 3 Blocks		UNIT:	D-8170 - Estimates		
			FILE:			

PAY ITEM	DESCRIPTION	CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
	Mobilization (at 5% of other items)		1	ls	$250,000.00	$250,000
	Furnish Cofferdam		1	ea	$500,000.00	$500,000
	Install & Move Cofferdam (h=70'; w= 50')		3	ea	$175,000.00	$525,000
	Excavation - concrete		88	cy	$1,400.00	$123,200
	Surface Preparation of Concrete		9,350	sf	$8.00	$74,800
	Sawcut (3" deep)		680	lf	$13.00	$8,840
	Mobilize Barges (1 large, 1 transit)		1	ls	$100,000.00	$100,000
	Mobilize Cranes (1 shore; 1 barge)		1	ls	$40,000.00	$40,000
	Barge Rental		9	mo	$35,000.00	$315,000
	Crane Rate (operated), large barge crane & smaller shore crane		9	mo	$85,000.00	$765,000
	Drilling for #11 anchor bars (2" dia holes)		19,700	lf	$24.00	$472,800
	Concrete:					
	Reinforced - face of dam		3,630	cy	$300.00	$1,089,000
	Furnishing & Handling Cementitious Material		1,030	tons	$120.00	$123,600
	Furnish & Install Reinforcement Bars		785,000	lbs	$0.65	$510,250
	Unwatering		3	ea	$10,000.00	$30,000
	Dewatering		9	mo	$30,000.00	$270,000
	Subtotal without mobilization					$4,947,490
	Subtotal with mobilization					$5,197,490
	Unlisted Items; 10% (+ or -)					$502,510
	Construction Cost					**$5,700,000**
	Contingencies; 20% (+ or -)					$1,200,000
	FIELD COST					**$6,900,000**

	QUANTITIES		PRICES			
BY	B Cohen	BY	Bill Holbert	CHECKED		
					Craig A. Grush 7/27/00	
DATE PREPARED		APPROVED	DATE	07/27/00	PRICE LEVEL	Feasibility
		E Hall				

CODE:D-8130			ESTIMATE WORKSHEET					Page 4 of 10
FEATURE:			29-Aug-00	PROJECT:				
		Grand Coolee Dam			**Columbia Basin Project**			
		Total Disolved Gas Study						
		Deflectors on face of Spillway		DIVISION:		Civil Engineering		
		68-ft Radius, Price is for 4 Blocks		UNIT:		D-8170 - Estimates		
				FILE:				
PAY							UNIT	
ITEM	DESCRIPTION			CODE	QUANTITY	UNIT	PRICE	AMOUNT
	Mobilization (at 5% of other items)				1	ls	$310,000.00	$310,000
	Furnish Cofferdam				1	ea	$500,000.00	$500,000
	Install & Move Cofferdam (h=70'; w= 50')				4	ea	$175,000.00	$700,000
	Excavation - concrete				117	cy	$1,300.00	$152,100
	Surface Preparation of Concrete				12,470	sf	$7.00	$87,290
	Sawcut (3" deep)				900	lf	$12.00	$10,800
	Mobilize Barges (1 large, 1 transit)				1	ls	$100,000.00	$100,000
	Mobilize Cranes (1 shore; 1 barge)				1	ls	$40,000.00	$40,000
	Barge Rental				12	mo	$35,000.00	$420,000
	Crane Rate (operated), large barge crane & smaller shore crane				12	mo	$85,000.00	$1,020,000
	Drilling for #11 anchor bars (2" dia holes)				26,200	lf	$22.00	$576,400
	Concrete:							
		Reinforced - face of dam			4,840	cy	$290.00	$1,403,600
	Furnishing & Handling Cementitious Material				1,370	tons	$120.00	$164,400
	Furnish & Install Reinforcement Bars				1,047,000	lbs	$0.65	$680,550
	Unwatering				4	ea	$10,000.00	$40,000
	Dewatering				12	mo	$30,000.00	$360,000
			Subtotal without mobilization					$6,255,140
			Subtotal with mobilization					$6,565,140
	Unlisted Items; 10% (+ or -)							$634,860
	Construction Cost							**$7,200,000**
	Contingencies; 20% (+ or -)							$1,500,000
	FIELD COST							**$8,700,000**
		QUANTITIES			PRICES			
BY	B Cohen			BY	Bill Holbert	CHECKED		
							Craig A. Grush 7/27/00	
DATE PREPARED			APPROVED	DATE	07/27/00	PRICE LEVEL		Feasibility
			E Hall					

ESTIMATE WORKSHEET					Page 5 of 10
FEATURE:	29-Aug-00	PROJECT:			
Grand Coolee Dam		**Columbia Basin Project**			
Total Disolved Gas Study					
Deflectors on face of Spillway		DIVISION:	Civil Engineering		
68-ft Radius, Price is for 5 Blocks		UNIT:	D-8170 - Estimates		
		FILE:			

PAY ITEM	DESCRIPTION	CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
	Mobilization (at 5% of other items)		1	ls	$370,000.00	$370,000
	Furnish Cofferdam		1	ea	$500,000.00	$500,000
	Install & Move Cofferdam (h=70'; w= 50')		5	ea	$175,000.00	$875,000
	Excavation - concrete		146	cy	$1,200.00	$175,200
	Surface Preparation of Concrete		15,590	sf	$7.00	$109,130
	Sawcut (3" deep)		1,130	lf	$11.00	$12,430
	Mobilize Barges (1 large, 1 transit)		1	ls	$100,000.00	$100,000
	Mobilize Cranes (1 shore; 1 barge)		1	ls	$40,000.00	$40,000
	Barge Rental		15	mo	$35,000.00	$525,000
	Crane Rate (operated), large barge crane & smaller shore crane		15	mo	$85,000.00	$1,275,000
	Drilling for #11 anchor bars (2" dia holes)		32,800	lf	$20.00	$656,000
	Concrete:					
	Reinforced - face of dam		6,050	cy	$280.00	$1,694,000
	Furnishing & Handling Cementitious Material		1,710	tons	$110.00	$188,100
	Furnish & Install Reinforcement Bars		1,309,000	lbs	$0.60	$785,400
	Unwatering		5	ea	$10,000.00	$50,000
	Dewatering		15	mo	$30,000.00	$450,000
	Subtotal without mobilization					$7,435,260
	Subtotal with mobilization					$7,805,260
	Unlisted Items; 10% (+ or -)					$794,740
	Construction Cost					**$8,600,000**
	Contingencies; 20% (+ or -)					$1,900,000
	FIELD COST					**$10,500,000**

	QUANTITIES		PRICES		
BY	B Cohen	BY	Bill Holbert	CHECKED	
				Craig A. Grush 7/27/00	
DATE PREPARED	APPROVED	DATE	07/27/00	PRICE LEVEL	Feasibility
	E Hall				

FEATURE:		29-Aug-00	PROJECT:				
	Grand Coolee Dam			**Columbia Basin Project**			
	Total Disolved Gas Study						
	Deflectors on face of Spillway		DIVISION:		Civil Engineering		
	68-ft Radius, Price is for 6 Blocks		UNIT:		D-8170 - Estimates		
			FILE:				
PAY						UNIT	
ITEM	DESCRIPTION		CODE	QUANTITY	UNIT	PRICE	AMOUNT
	Mobilization (at 5% of other items)			1	ls	$430,000.00	$430,000
	Furnish Cofferdam			1	ea	$500,000.00	$500,000
	Install & Move Cofferdam (h=70'; w= 50')			6	ea	$175,000.00	$1,050,000
	Excavation - concrete			175	cy	$1,100.00	$192,500
	Surface Preparation of Concrete			18,700	sf	$7.00	$130,900
	Sawcut (3" deep)			1,350	lf	$10.00	$13,500
	Mobilize Barges (1 large, 1 transit)			1	ls	$100,000.00	$100,000
	Mobilize Cranes (1 shore; 1 barge)			1	ls	$40,000.00	$40,000
	Barge Rental			18	mo	$35,000.00	$630,000
	Crane Rate (operated), large barge crane & smaller shore crane			18	mo	$85,000.00	$1,530,000
	Drilling for #11 anchor bars (2" dia holes)			39,300	lf	$18.00	$707,400
	Concrete:						
		Reinforced - face of dam		7,250	cy	$275.00	$1,993,750
	Furnishing & Handling Cementitious Material			2,050	tons	$110.00	$225,500
	Furnish & Install Reinforcement Bars			1,570,000	lbs	$0.60	$942,000
	Unwatering			6	ea	$10,000.00	$60,000
	Dewatering			18	mo	$30,000.00	$540,000
			Subtotal without mobilization				$8,655,550
		Subtotal with mobilization					$9,085,550
	Unlisted Items; 10% (+ or -)						$914,450
	Construction Cost						**$10,000,000**
	Contingencies; 20% (+ or -)						$2,000,000
	FIELD COST						**$12,000,000**
	QUANTITIES			PRICES			
BY	B Cohen		BY	Bill Holbert	CHECKED		
						Craig A. Grush 7/27/00	
DATE PREPARED		APPROVED	DATE	07/27/00	PRICE LEVEL		Feasibility
		E Hall					

CODE:D-8130		ESTIMATE WORKSHEET				Page 7 of 10
FEATURE:		29-Aug-00	PROJECT:			
	Grand Coolee Dam		**Columbia Basin Project**			
	Total Disolved Gas Study					
	Deflectors on face of Spillway		DIVISION:	Civil Engineering		
	68-ft Radius, Price is for 7 Blocks		UNIT:	D-8170 - Estimates		
			FILE:			

PAY ITEM	DESCRIPTION	CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
	Mobilization (at 5% of other items)		1	ls	$490,000.00	$490,000
	Furnish Cofferdam		1	ea	$500,000.00	$500,000
	Install & Move Cofferdam (h=70'; w= 50')		7	ea	$175,000.00	$1,225,000
	Excavation - concrete		205	cy	$1,100.00	$225,500
	Surface Preparation of Concrete		21,820	sf	$7.00	$152,740
	Sawcut (3" deep)		1,580	lf	$9.00	$14,220
	Mobilize Barges (1 large, 1 transit)		1	ls	$100,000.00	$100,000
	Mobilize Cranes (1 shore; 1 barge)		1	ls	$40,000.00	$40,000
	Barge Rental		21	mo	$35,000.00	$735,000
	Crane Rate (operated), large barge crane & smaller shore crane		21	mo	$85,000.00	$1,785,000
	Drilling for #11 anchor bars (2" dia holes)		45,900	lf	$16.00	$734,400
	Concrete:					
	Reinforced - face of dam		8,460	cy	$270.00	$2,284,200
	Furnishing & Handling Cementitious Material		2,400	tons	$110.00	$264,000
	Furnish & Install Reinforcement Bars		1,832,000	lbs	$0.55	$1,007,600
	Unwatering		7	ea	$10,000.00	$70,000
	Dewatering		21	mo	$30,000.00	$630,000
			Subtotal without mobilization			$9,767,660
		Subtotal with mobilization				$10,257,660
	Unlisted Items; 10% (+ or -)					$1,242,340
	Construction Cost					**$11,500,000**
	Contingencies; 20% (+ or -)					$2,000,000
	FIELD COST					**$13,500,000**
	QUANTITIES		PRICES			
BY	B Cohen	BY	Bill Holbert	CHECKED		
					Craig A. Grush 7/27/00	
DATE PREPARED		APPROVED	DATE 07/27/00	PRICE LEVEL		Feasibility
		E Hall				

CODE:D-8130	ESTIMATE WORKSHEET					Page 8 of 10
FEATURE:	29-Aug-00	PROJECT:				
	Grand Coolee Dam		**Columbia Basin Project**			
	Total Disolved Gas Study					
	Deflectors on face of Spillway	DIVISION:		Civil Engineering		
	68-ft Radius, Price is for 8 Blocks	UNIT:		D-8170 - Estimates		
		FILE:				

PAY ITEM	DESCRIPTION	CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
	Mobilization (at 5% of other items)		1	ls	$540,000.00	$540,000
	Furnish Cofferdam		1	ea	$500,000.00	$500,000
	Install & Move Cofferdam (h=70'; w= 50')		8	ea	$175,000.00	$1,400,000
	Excavation - concrete		234	cy	$1,000.00	$234,000
	Surface Preparation of Concrete		24,940	sf	$7.00	$174,580
	Sawcut (3" deep)		1,800	lf	$8.00	$14,400
	Mobilize Barges (1 large, 1 transit)		1	ls	$100,000.00	$100,000
	Mobilize Cranes (1 shore; 1 barge)		1	ls	$40,000.00	$40,000
	Barge Rental		24	mo	$35,000.00	$840,000
	Crane Rate (operated), large barge crane & smaller shore crane		24	mo	$85,000.00	$2,040,000
	Drilling for #11 anchor bars (2" dia holes)		52,400	lf	$14.00	$733,600
	Concrete:					
	Reinforced - face of dam		9,670	cy	$260.00	$2,514,200
	Furnishing & Handling Cementitious Material		2,740	tons	$110.00	$301,400
	Furnish & Install Reinforcement Bars		2,094,000	lbs	$0.55	$1,151,700
	Unwatering		8	ea	$10,000.00	$80,000
	Dewatering		24	mo	$30,000.00	$720,000
	Subtotal without mobilization					$10,843,880
	Subtotal with mobilization					$11,383,880
	Unlisted Items; 10% (+ or -)					$1,116,120
	Construction Cost					**$12,500,000**
	Contingencies; 20% (+ or -)					$2,500,000
	FIELD COST					**$15,000,000**

	QUANTITIES		PRICES			
BY	B Cohen	BY	Bill Holbert	CHECKED		
					Craig A. Grush 7/27/00	
DATE PREPARED	APPROVED	DATE	07/27/00	PRICE LEVEL		Feasibility
	E Hall					

CODE:D-8130			ESTIMATE WORKSHEET					Page 9 of 10
FEATURE:			29-Aug-00	PROJECT:				
		Grand Coolee Dam			**Columbia Basin Project**			
		Total Disolved Gas Study						
		Deflectors on face of Spillway		DIVISION:		Civil Engineering		
		68-ft Radius, Price is for 9 Blocks		UNIT:		D-8170 - Estimates		
				FILE:				
PAY							UNIT	
ITEM	DESCRIPTION			CODE	QUANTITY	UNIT	PRICE	AMOUNT
	Mobilization (at 5% of other items)				1	ls	$590,000.00	$590,000
	Furnish Cofferdam				1	ea	$500,000.00	$500,000
	Install & Move Cofferdam (h=70'; w= 50')				9	ea	$175,000.00	$1,575,000
	Excavation - concrete				263	cy	$1,000.00	$263,000
	Surface Preparation of Concrete				28,050	sf	$7.00	$196,350
	Sawcut (3" deep)				2,030	lf	$7.00	$14,210
	Mobilize Barges (1 large, 1 transit)				1	ls	$100,000.00	$100,000
	Mobilize Cranes (1 shore; 1 barge)				1	ls	$40,000.00	$40,000
	Barge Rental				27	mo	$35,000.00	$945,000
	Crane Rate (operated), large barge crane & smaller shore crane				27	mo	$85,000.00	$2,295,000
	Drilling for #11 anchor bars (2" dia holes)				59,000	lf	$12.00	$708,000
	Concrete:							
		Reinforced - face of dam			10,880	cy	$250.00	$2,720,000
	Furnishing & Handling Cementitious Material				3,080	tons	$100.00	$308,000
	Furnish & Install Reinforcement Bars				2,355,000	lbs	$0.50	$1,177,500
	Unwatering				9	ea	$10,000.00	$90,000
	Dewatering				27	mo	$30,000.00	$810,000
			Subtotal without mobilization					$11,742,060
			Subtotal with mobilization					$12,332,060
	Unlisted Items; 10% (+ or -)							$1,167,940
	Construction Cost							**$13,500,000**
	Contingencies; 20% (+ or -)							$3,000,000
	FIELD COST							**$16,500,000**
		QUANTITIES			PRICES			
BY	B Cohen			BY	Bill Holbert	CHECKED		
							Craig A. Grush 7/27/00	
DATE PREPARED			APPROVED	DATE	07/27/00	PRICE LEVEL		Feasibility
			E Hall					

CODE:D-8130			ESTIMATE WORKSHEET					Page 10 of 10
FEATURE:			29-Aug-00	PROJECT:				
		Grand Coolee Dam			**Columbia Basin Project**			
		Total Disolved Gas Study						
		Deflectors on face of Spillway		DIVISION:		Civil Engineering		
		68-ft Radius, Price is for 10 Blocks		UNIT:		D-8170 - Estimates		
				FILE:				
PAY							UNIT	
ITEM	DESCRIPTION			CODE	QUANTITY	UNIT	PRICE	AMOUNT
	Mobilization (at 5% of other items)				1	ls	$650,000.00	$650,000
	Furnish Cofferdam				1	ea	$500,000.00	$500,000
	Install & Move Cofferdam (h=70'; w= 50')				10	ea	$175,000.00	$1,750,000
	Excavation - concrete				292	cy	$1,500.00	$438,000
	Surface Preparation of Concrete				31,170	sf	$7.00	$218,190
	Sawcut (3" deep)				2,250	lf	$6.00	$13,500
	Mobilize Barges (1 large, 1 transit)				1	ls	$100,000.00	$100,000
	Mobilize Cranes (1 shore; 1 barge)				1	ls	$40,000.00	$40,000
	Barge Rental				30	mo	$35,000.00	$1,050,000
	Crane Rate (operated), large barge crane & smaller shore crane				30	mo	$85,000.00	$2,550,000
	Drilling for #11 anchor bars (2" dia holes)				65,500	lf	$10.00	$655,000
	Concrete:							
		Reinforced - face of dam			12,090	cy	$250.00	$3,022,500
	Furnishing & Handling Cementitious Material				3,420	tons	$100.00	$342,000
	Furnish & Install Reinforcement Bars				2,617,000	lbs	$0.50	$1,308,500
	Unwatering				10	ea	$10,000.00	$100,000
	Dewatering				30	mo	$30,000.00	$900,000
			Subtotal without mobilization					$12,987,690
			Subtotal with mobilization					$13,637,690
	Unlisted Items; 10% (+ or -)							$1,362,310
	Construction Cost							**$15,000,000**
	Contingencies; 20% (+ or -)							$3,000,000
	FIELD COST							**$18,000,000**
		QUANTITIES			PRICES			
BY	B Cohen			BY	Bill Holbert	CHECKED		
							Craig A. Grush 7/27/00	
DATE PREPARED			APPROVED	DATE	07/27/00	PRICE LEVEL		Feasibility
			E Hall					

APPENDIX 3

Estimate Worksheet for Forebay Cascade

ODE:D-8130	ESTIMATE WORKSHEET					Page 1 of 2
EATURE:	14-JUL-00	PROJECT:				
	Grand Coolee Dam		**Columbia Basin Project**			
	Total Disolved Gas Study					
	Forebay Cascade - Alt-5	DIVISION:		Civil Engineering		
		UNIT:		D-8170 - Estimates		
		FILE:				
PAY ITEM	DESCRIPTION	CODE	QUANTITY	UNIT	UNIT PRICE	AMOUNT
	Mobilization (@ 5%)		1	LS	$10,500,000.00	$10,500,000
	Furnish and Install Cellular Cofferdam; H=180'; L=220'		1	LS	$15,000,000.00	$15,000,000
						$0
	Furnish and Install Cellular Cofferdam; H=40'; L=1520'		1	LS	$8,000,000.00	$8,000,000
						$0
	Concrete:					$0
	Reinforced: Chamber structure		102,000	CY	$250.00	$25,500,000
	Reinforced: Baffles and chute blocks		3,000	CY	$275.00	$825,000
	Reinforced: Floating slab		9,000	CY	$200.00	$1,800,000
	Reinforced: Piers		2,000	CY	$350.00	$700,000
	Reinforced: Tunnel (54 ft dia tunnel)		18,500	CY	$225.00	$4,162,500
	Mass Concrete upstream		160,000	CY	$130.00	$20,800,000
	Mass Concrete downstream		43,000	CY	$160.00	$6,880,000
						$0
	Furnishing and handling cementitious materials		76,000	tons	$110.00	$8,360,000
						$0
	Furnishing and installing reinforcing bars		20,000,000	lbs	$0.65	$13,000,000
						$0
	Furnishing and installing 40'-0" dia steel pipe		16,000,000	lbs	$1.50	$24,000,000
	Furnish and install 40'-0" dia wheel mounted gate		615,000	lbs	$6.00	$3,690,000
	Furnishing and installing regulating gates and operators		1,100,000	lbs	$7.00	$7,700,000
						$0
	Furnishing and installing hoists, wire rope and accessories		195,000	lbs	$6.00	$1,170,000
						$0
	Furnishing and installing floating stilling basin metal work		31,000,000	lbs	$1.50	$46,500,000
						$0
	Furnish and install trash rack		600,000	lbs	$2.50	$1,500,000
						$0
	Unwatering		1	ls	$1,000,000.00	$1,000,000
						$0
	Dewatering		1	ls	$3,000,000.00	$3,000,000
						$0
	Crane Mobilization		1	ls	$15,000.00	$15,000
						$0
	Barge Mobilization		2	ls	$25,000.00	$50,000
						$0
	Barge Rent		8	mo	$85,000.00	$680,000
						$0
	Crane Rent		8	mo	$80,000.00	$640,000
	QUANTITIES		PRICES			
Y	B Cohen	BY	Bill Holbert	CHECKED		
					RKC 07/14/00	
ATE PREPARED	APPROVED	DATE	07/14/00	PRICE LEVEL		Feasibility
	E Hall					

CODE:D-8130			ESTIMATE WORKSHEET					
FEATURE:			14-JUL-00	PROJECT:				
		Grand Coolee Dam			**Columbia Basin Project**			
		Total Disolved Gas Study						
		Forebay Cascade - Alt-5		DIVISION:		Civil Engineering		
				UNIT:		D-8170 - Estimates		
			Page 2	FILE:				
PAY							UNIT	
ITEM		DESCRIPTION		CODE	QUANTITY	UNIT	PRICE	AMOUNT
	Excavation - common				650,000	cy	$4.50	$2,925,000
								$0
	Excavation - rock				345,000	cy	$15.00	$5,175,000
	Drilling for piers (5' dia in rock)				700	cy		$0
	48 holes - 20' deep				960	lf	$2,600.00	$2,496,000
								$0
	Excavation (rock Tunnel)				34,000	cy	$175.00	$5,950,000
								$0
	Compacted backfill				330,000	cy	$5.00	$1,650,000
								$0
	Furnish & Install ventilation system consisting of 4000 cfm				1	ls	$85,000.00	$85,000
	centrifugal fan and 100 feet of 18" dia schedule 10							$0
	carbon steel pipe and 660 feet of 18" dia schedule 40 PVC pipe							$0
								$0
	Furnish and Install on e geared electric traction freight				1	ls	$150,000.00	$150,000
	elevator with a capacity of 3,500 lbs. Elevator							$0
	will have two landings with a car size of 8'-0" x 8'-0"							$0
	and a total travel of 137 feet.							$0
			Subtotal before mobilization					$213,403,500
			Subtotal with mobilization					$223,903,500
	Unlisted Items (10% + or -)							$26,096,500
	Contract Cost							**$250,000,000**
	Contingencies (20% + or -)							$50,000,000
	FIELD COST							**$300,000,000**
		QUANTITIES			PRICES			
BY	B Cohen			BY	Bill Holbert	CHECKED		
							RKC 07/14/00	
DATE PREPARED			APPROVED	DATE	07/14/00	PRICE LEVEL		Feasibility
			E Hall					