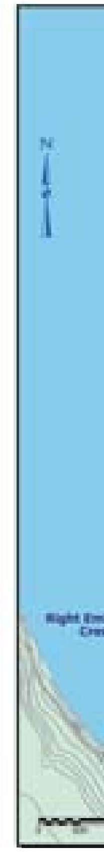
STABILITY REVIEW OF THE WANAPUM SPILLWAY USING CFD ANALYSIS



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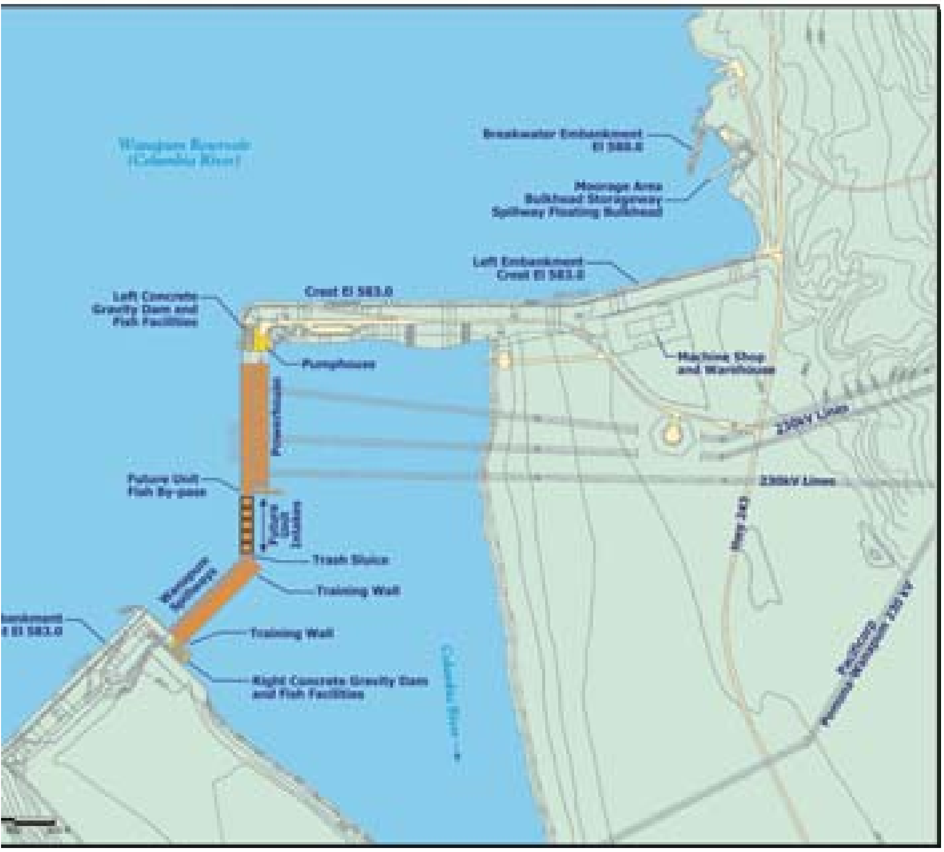
The Wanapum Dam is located in the State of Washington on the Columbia River, approximately four miles downstream of I-90 and the town of Vantage (Figure 1). The layout for the project is shown in Figure 2. Completed in 1964, the Wanapum development consists of an earthen embankment on the left abutment, a left gravity section with fish facilities, a powerhouse with 10 generation units, six future unit intakes, 12 spillway monoliths with large radial gates, a right gravity section with fish facilities, and another earthen embankment on the right abutment.

The discharge facilities for the project consist of 12 spillway bays with 50-foot (15.3 m) wide by 67-foot (20.4 m) high radial gates supported by 15 foot (4.6 m) wide piers. At the time of construction in the early 1960s they were the largest radial gates ever installed and the first to utilize post tension anchors for the trunnion anchorages. The overall length of the spillway monolith is 110 feet (33.5 m) and the structure includes a 69-foot (21.0 m) long apron complete with an end sill. The spillway crest is set at El. 505 ft (153.9 m),



Above: Figure 2 - Project Layout

Left: Figure 1 - Project Location

Le barrage Wanapum se trouve sur le fleuve Columbia, dans l'État de Washington, à quatre milles en aval de l'autoroute I-90 et de la ville de Vantage. Ce projet, achevé en 1964, se compose d'un endiguement de terre sur la culée gauche, d'une culée-poids gauche avec des installations pour les poissons, d'une centrale avec 10 turbines, six points de prélèvement pour des turbines futures, 12 blocs de déversement avec de grandes vannes à segment, une culée-poids droite avec des installations pour les poissons et un autre endiguement de terre sur la culée droite.

Conformément aux recommandations de la FERC, partie 12D sur les inspections de sécurité, Hatch Acres a reçu le mandat d'examiner le barrage Wanapum Dam dans des conditions de charge qui comprenaient la crue maximale probable (CMP). Un premier examen des analyses antérieures a révélé que les analyses de stabilité avaient peut-être sous-estimé les forces de crête.

Les forces hydrodynamiques agissant sur un radier et un déversoir en temps de crue viennent du changement de vitesse et de direction de l'eau alors qu'elle s'écoule sur le déversoir. La plus récente édition des règles de la FERC pour les barrages-poids inclut une nouvelle méthode d'évaluation des forces hydrodynamiques s'exerçant sur le déversoir lorsque l'écoulement est supérieur au débit de calcul.

Grâce aux récents progrès de la dynamique numérique des fluides (DNF), les modèles numériques offrent maintenant une méthode économique pour représenter les forces de lame déversante pour divers régimes, dont la CMP. L'évaluation d'une gamme de débits permet de confirmer les conditions critiques du point de vue de la stabilité structurale. Cet article fait état des analyses de stabilité effectuées pour les blocs de déversement du barrage Wanapum sous diverses pressions modélisées par DNF. Il explore aussi l'influence des hypothèses d'efficacité de drain et de sous-pression, y compris l'effet de l'arrière-radier.

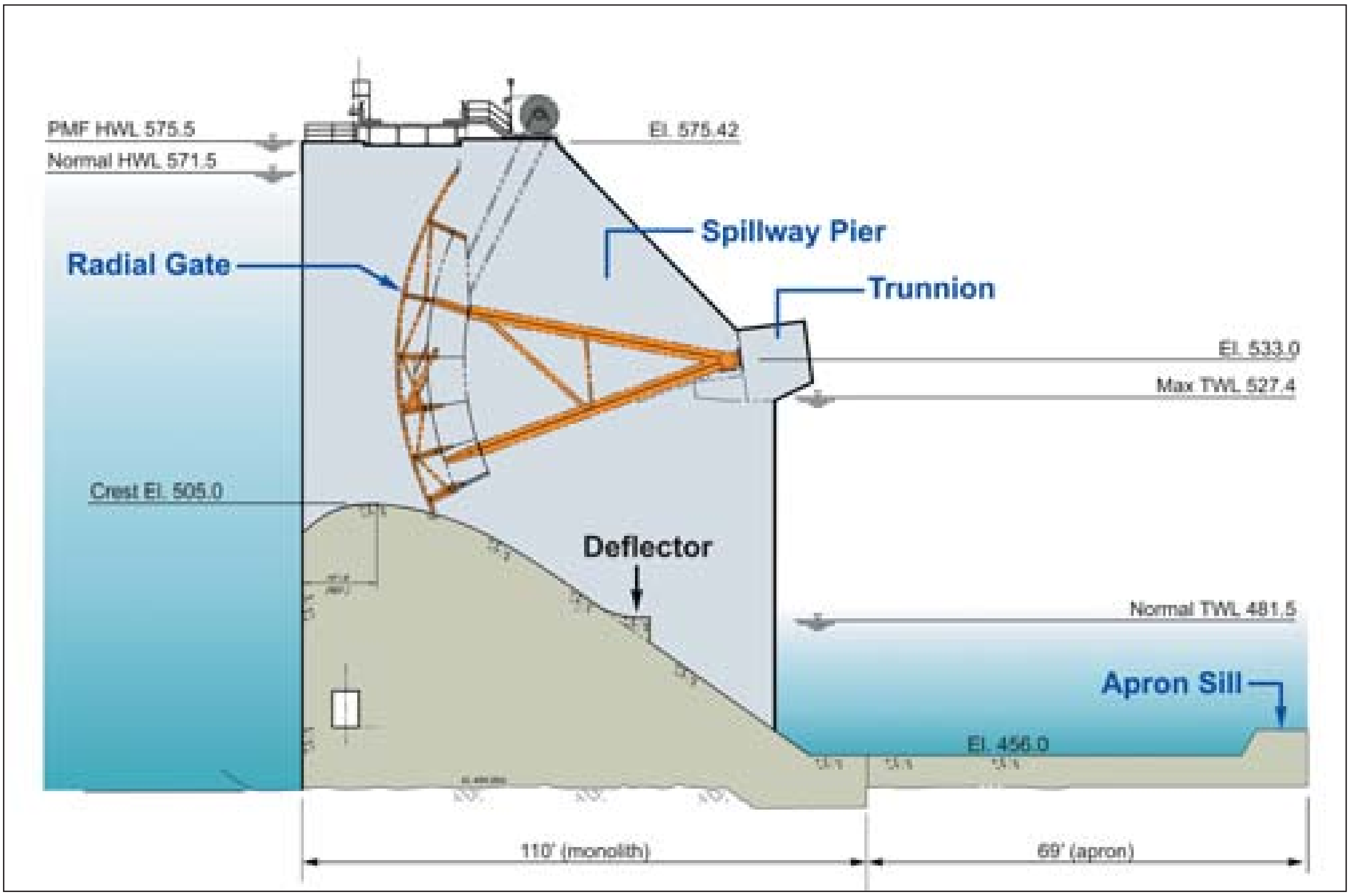


Figure 3 - Wanapum Spillway Section

and the base of the monolith is at El. 450 ft (137.2 m). The top of the six-foot thick apron is at El. 456 ft (139.0 m) and incorporates a four-foot (1.2 m) high end sill. Figure 3 shows a typical section through the spillway.

Hatch Acres was retained by Grant County PUD No. 2 to complete engineering assignments required as a follow-up engineering review after the U.S. Federal Energy Regulatory Commission (FERC) Part 12 D dam safety inspection completed in 2005. The FERC regulates and oversees energy industries including hydroelectric facilities in the US. They address economic, environmental, and safety interests of the American public.

One of Hatch Acres' assignments was to confirm the stability of the Wanapum spillway monoliths under a variety of load conditions. These load conditions included the passage of the Probable Maximum Flood (PMF), which is the design flow for the facility. Initial results indicated that previous analyses had not fully accounted for possible variations in hydrodynamic forces on the spillway flow surfaces under design discharge conditions.

The depth of flow over the Wanapum spillway bays would be significant during passage of the PMF, and it was not immediately clear what the pressure distribution on the ogee crest would be, or where the hydraulic jump would form – on or downstream of the apron sill. Therefore, previous stability analyses had used a variety of assumptions ranging from full water weight over the submerged spillway to no water weight and 60% of the tailwater height above the apron to resist sliding. This led to a significant variation in previous study results.

Hydrodynamic forces will begin to act on a spillway and apron during passage of a flood due to the change in the speed and direction of the water as it passes over the spillway. The 2002 FERC engineering guidelines for gravity dams include a revision to the manner in which nappe forces on the spillway are to be evaluated for flows greater than the design discharge:

"The forces acting on an overflow dam or spillway section are complicated by steady state hydrodynamic effects. Hydrodynamic forces result from water changing speed and direction as it flows over a spillway. At small discharges, nappe forces may be neglected in stability analysis; however, when the discharge over an overflow spillway approaches the design discharge, nappe forces can become significant and should be taken into account in the analysis of dam stability."

Accordingly, the Hatch Acres team sought to fully integrate pressure profile data into the stability calculations for the structure. This involved the development of a two-dimensional CFD model, and the application of this model to estimate nappe forces on the profile. These calculated forces were then incorporated into stability analyses carried out to check spillway stability during passage of the PMF. The stability analyses for the PMF condition were first carried out using a conventional analysis

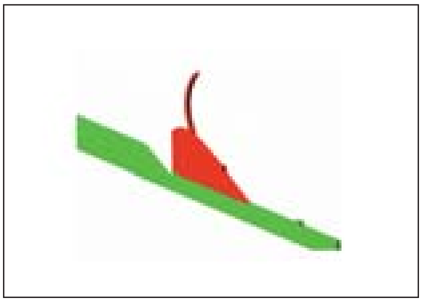


Figure 4 - CFD Model

approach, and secondly by using CFD generated nappe forces. The results for the two were compared and are presented in this article.

COMPUTATIONAL FLUID DYNAMICS ASSESSMENT

With the advancements in computing power made since the 1980s, Computational Fluid Dynamics (CFD) analysis has emerged as a powerful hydraulics design tool. CFD was selected for this study to provide additional insight into the nature of the pressure distribution over the crest of the Wanapum Spillway, and how it may vary for full as well as partial gate openings.

Selected Model - FLOW-3D

The FLOW-3D computer model, developed by Flow Science Incorporated of Los Alamos, New Mexico was selected for use in this study. FLOW-3D has been designed for the treatment of time-dependent (transient) problems in one, two and three dimensions, and is based on a solution of the complete Navier Stokes equations. Because the program is based on the fundamental laws of mass, momentum and energy conservation, it is applicable to almost any type of flow process. For this reason, FLOW-3D is often referred to as a "general purpose" CFD solver.

One of the major strengths of the FLOW-3D program for hydraulic analysis is its ability to accurately model problems involving free surface flows. An interface between a gas and a liquid is referred to as a free surface. In FLOW-3D, free surfaces are modeled with the Volume of Fluid (VOF) technique. The VOF method consists of three ingredients: a scheme to locate the surface, an algorithm to track the surface as a sharp interface moving through a computational grid, and a means of applying boundary conditions at the surface.

FLOW-3D uses a simple grid of rectangular elements so it has the advantages of ease of generation, regularity for improved numerical accuracy, and requires minimal memory storage. Geometry is then defined within the grid by computing the fractional face areas and fractional volumes of each element that are blocked by obstacles. The equations of motion are then solved based on a finite difference technique.

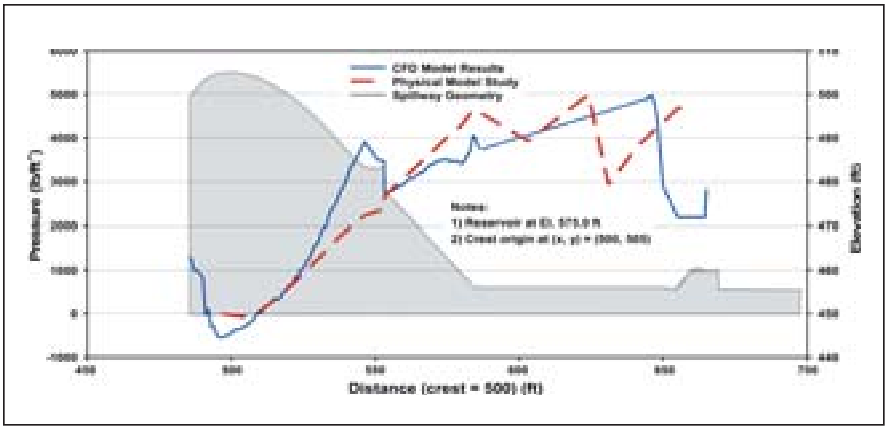


Figure 5 - Pressure Graph under PMF conditions

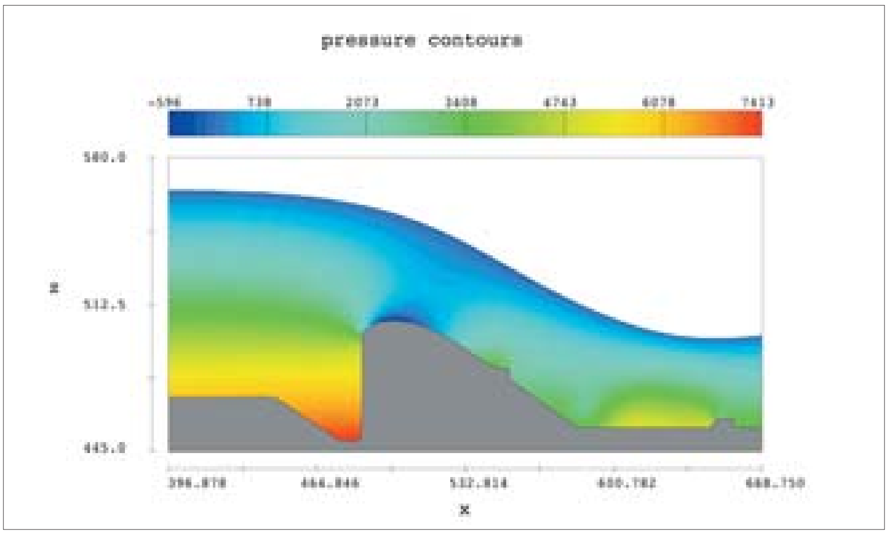


Figure 6 - Pressure Contours for PMF (lb/ft^2)

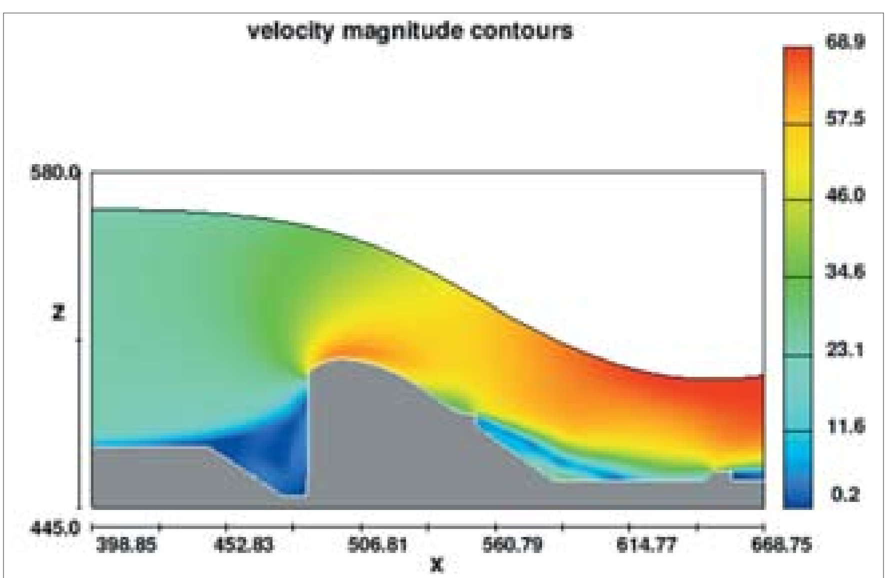


Figure 7 - Velocity Contours for PMF (ft/s)

CFD Model Setup

Since all 12 spillway bays are of an identical size and configuration, a two-dimensional numerical model representing a unit width of the spillway was developed to analyze the hydraulic pressure distribution over the spillway structure. A sectional model was run in lieu of a full three-dimensional simulation in order to help minimize overall computational effort, and thereby allow maximum resolution of the computational mesh over the ogee section. Three different discharge conditions were tested to simulate a range of possible hydraulic pressures and forces expected on the spillway monolith. However, only the results of the PMF simulations are presented in this article.

The developed numerical model included the Wanapum Dam spillway, its stilling basin, and associated approach and tailrace channels. The model extended from a point approximately 450 ft (137.2 m) upstream of the spillway to a point approximately 300 ft (91.4 m) downstream of the apron sill. Care was taken in selecting the upstream and downstream boundary conditions of the model to ensure that entrance condition and approach losses and the tailrace water level impacts were being reasonably simulated.

Information used in the "construction" of the model was gathered from a number of sources including past construction drawings and earlier study reports. The final physical representation of the spillway as a unit "slice" at the center of a bay is shown in Figure 4. The model was built entirely within AutoCAD, and was then imported into the numerical model as a stereolithographic (STL) file.

The upstream and downstream boundaries for the model were set as a prescribed elevation boundary. The model then automatically calculates flows through the spillway based on the prescribed boundaries, and the spillway geometry. The numerical model was run for just over eight minutes (prototype time) to simulate each scenario. This time frame was sufficient for the model to achieve convergence and each run took approximately 1.5 hours of computer time on a 3.2 GHz Pentium 4 workstation with 2GB of RAM.

The PMF flow for the Wanapum Dam was previously estimated to be 1,400,000 cfs (39640 m^3/s). During the PMF, the gates were assumed to be fully open and the powerhouse was assumed not to be operating. The model was set up based on the expected reservoir and tailwater levels of 575.0 ft (175.25 m) and 527.2 ft (160.7 m) respectively.

This scenario simulated passage of the PMF event – the design flow for this facility. It also provided an excellent opportunity to compare the results of the numerical model with those of earlier physical model studies conducted prior to its construction in the early 1960s. This was an important step as it allowed validation of the numerical model's ability to replicate the hydraulic performance of this structure.

CFD Model Results

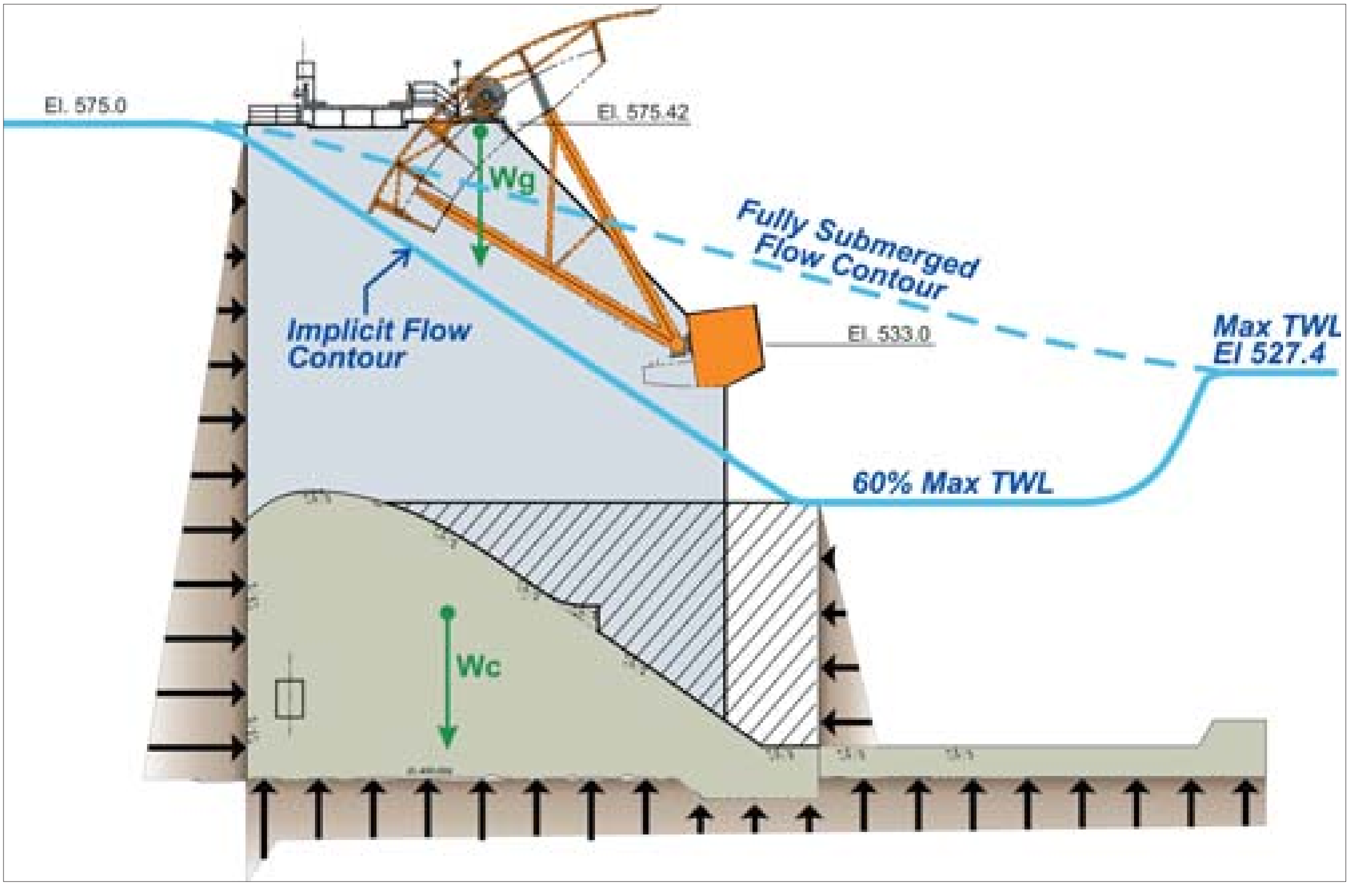
The CFD model was run until a steady flow was achieved through the spillway structure. The final discharge achieved, based on a headwater elevation of 575 ft (175.25 m), was 1,400,000 cfs (39640 m^3/s) with the tainter gates fully opened. This compared exactly with discharge estimates provided by the physical model studies which were conducted prior to the construction of the dam. Figure 5 compares the pressure distribution over the spillway as determined from both the CFD Model and the earlier Physical Model Study.

The physical model and CFD analysis appear to have good agreement, especially if the physical model study results were to be smoothed. The physical model did not include the deflector or the apron sill. This is reflected in the higher pressures predicted by CFD at the deflector location and the differences in predicted pressures at the apron sill location.

Figure 6 shows pressure contours over the spillway and apron for the PMF condition, while Figure 7 shows the corresponding velocity profile. As shown, the maximum hydrodynamic pressure on the spillway monolith would be 7,340 lb/ft^2 (352 kN/m^2), at a location just upstream of the vertical face of the spillway. This is about 6% less than hydrostatic pressure with the reservoir at elevation 575.0 ft (175.25 m). Negative pressures would occur over a section of the crest extending from a point 10 ft (3.0 m) upstream of the crest to a point 10 ft (3.0 m) downstream. The minimum (negative) pressure was computed to be -556 lb/ft^2 (-27.0 kN/m^2), and would occur very near to the spillway crest. These CFD results under PMF are consistent with earlier physical model study tests that were performed in 1964.

Under PMF conditions, the CFD results showed that although the hydraulic jump would begin to form immediately downstream of the structure, full tailwater would not be established until 200 to 250 ft (61.0 to 76.0 m) downstream of the apron sill. This was con-

Figure 8 - PMF Load diagram for Conventional Analysis



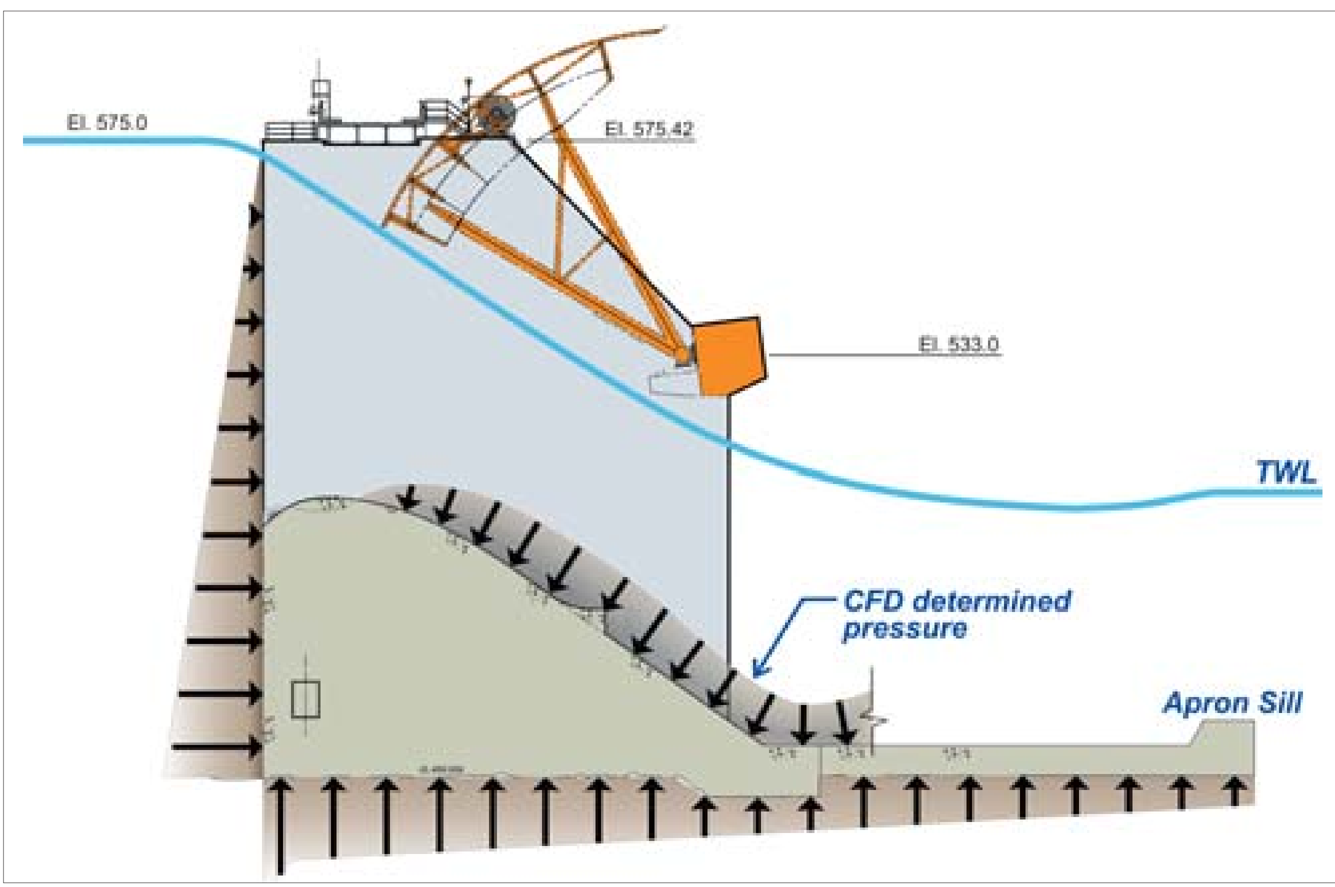


Figure 9 - PMF Load diagram for CFD Based Analysis

sistent with the sectional physical model study observations.

STABILITY ANALYSIS RESULTS

Stability under PMF Condition

The stability of spillway structures is often sensitive to nappe pressure and tailwater elevation assumptions especially under high flow conditions. When performing conventional stability analysis under high flow conditions it is typically assumed that:

1. Water weight over the spillway is negligent because of potentially low or negative pressures on the nappe
2. Tailwater is 60% of expected height at the base of the spillway slope
3. Full tailwater hydrostatic pressure exists downstream of the apron establishing the downstream uplift pressure on the apron.

The 60% reduction in tailwater assumed in conventional analysis is intended to account for a possibility that a hydraulic jump might occur over the apron.

Figure 8 shows the assumed forces acting on the monolith for a conventional stability analysis. Analysis showed that the base of the spillway remained in compression under all cases considered so the uplift pressure distribution was not effected by crack propagation.

CFD analysis for the PMF condition provided estimated pressures at one-foot increments across the entire spillway crest and apron. The CFD pressures were integrated across the spillway monolith flow surface to determine the resultant magnitude and location.

Sliding safety factors were computed for a range of uplift pressures below the apron sill. The analysis was carried out for uplift pressures at the end of the apron varying from 50% and 100% of expected tailwater. CFD results indicated pressures downstream of the apron would be

TABLE 1

PMF Condition: SSF for CFD and Conventional Analysis

Apron Sill Uplift	CFD Based Analysis	Conventional Analysis
1. 60% of Max Tailwater	3.09	2.23
2. Full Maximum Tailwater	2.07	1.39

NOTES:

1. Assumed drain efficiency = 75% and friction angle (phi) = 59 degrees.

TABLE 2

PMF Condition: Minimum Required Drain Efficiency (SSF = 1.3)

Apron Sill Uplift	CFD Based Analysis	Conventional Analysis
1. 60% of Max Tailwater	0 % (2)	27% (3)
2. Full Maximum Tailwater	20%	67%

NOTES:

1. Assumed phi = 59 degrees
2. Minimum SSF = 1.42 with drain efficiency = 0.
3. Cracking of the base appears to start with drain efficiencies less than 26% for conventional analysis.

TABLE 3

PMF Condition: Minimum Required Phi Values (SSF = 1.3)

Apron Sill Uplift	CFD Based Analysis	Conventional Analysis
1. 60% of Max Tailwater	35°	45°
2. Full Maximum Tailwater	47°	58°

NOTES:

1. Assumed drain efficiency = 75%.

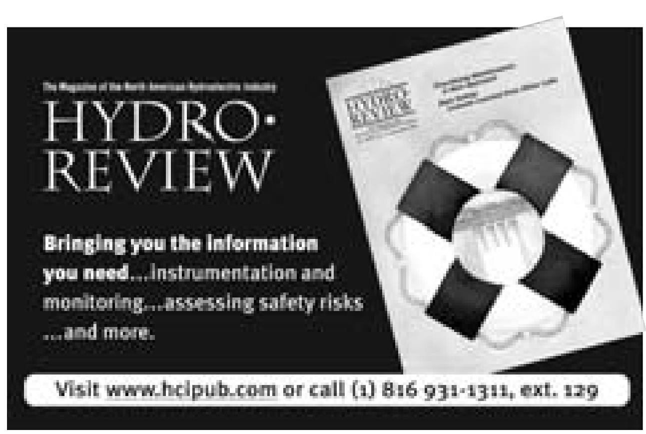
approximately 55% of expected tailwater. Under PMF the CFD results showed that although the hydraulic jump would begin to form immediately downstream of the structure, full tailwater is not established until 200 to 250 ft (61.0 to 76.0 m) downstream of the apron sill.

Figure 9 shows the assumed pressure distribution on the monolith and apron. Hydrostatic pressures were applied to the upstream vertical surfaces of the spillway monolith and to the base of the structure as uplift. As with the conventional analysis, it was determined that the entire spillway base remains in compression for the cases considered.

Stability analysis results for the spillway monolith alone are presented in Tables 1 through 3. The sliding safety factor (SSF) based on the CFD analysis was found to be approximately 40 to 50% higher than the sliding safety factor calculated using the conventional method. Table 1 shows the SSF assuming that the drain efficiency and friction angle are not reduced from the normal operating condition. If the apron weight, hydrodynamic pressure and uplift are included the SSF increase could be up to 30% higher than the SSF computed for the monolith alone. This increase is due to the pocket of high pressure that develops over the apron due to the change in flow direction. Inclusion of the apron in the PMF analysis may be justified in the case of Wanapum since the base remains in compression and the spillway monolith to apron joint should remain intact under a high flow event.

Table 2 shows the minimum drain efficiency required to achieve a sliding safety factor of 1.3 assuming the friction angle remains the same as under normal operating conditions. It is interesting to note that the much lower drain efficiencies are required to meet FERC requirements.

Table 3 shows the minimum phi values required to achieve the a sliding safety factor of 1.3 assuming the drain efficiency remains the same



as under normal operating conditions. Phi values can be significantly reduced from the value assumed under normal operating conditions and FERC minimum sliding safety factor of 1.3 can still be met.

SUMMARY

In summary, spillway nappe pressures were estimated successfully using computational fluid dynamics (CFD). Nappe pressures under the Probable Maximum Flood (PMF) produced a net positive contribution to the stability of the spillway monolith. PMF pressures along the ogee were slightly negative over 20 ft (6.1 m) but positive pressures dominated in the vicinity of the deflector and at the base of the spillway. The CFD results and model study results showed good agreement.

Stability analysis results for the PMF based on CFD pressures for the Wanapum spillway indicate that the conventional stability analysis approach may underestimate the sliding safety factor by 40-50% for the monolith alone. The authors recommend that CFD analysis be more widely used as a tool in the stability analysis of spillways under PMF and other flow conditions. It appears to provide an accurate estimate of pressures on spillway flow surfaces and of tailwater flow conditions and, in some cases, may eliminate the need for installation of anchors that might be found necessary based on conventional analysis. ■

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