Spillway design considerations when given hydro-static and hydro-dynamic loads

National Dam Safety Program Technical Seminar | 2024

Presenter: Lan Nguyen, Civil Engineer, Ph.D, P.E

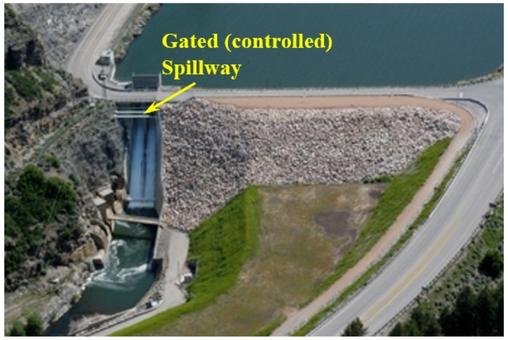
FEMA

Agenda

- Spillway structural components
- The loads, the magnification from static load to dynamic load.
- The responses of spillway structural component with respect to the loads
- Some examples and practice procedures when work on these assignments

Spillway

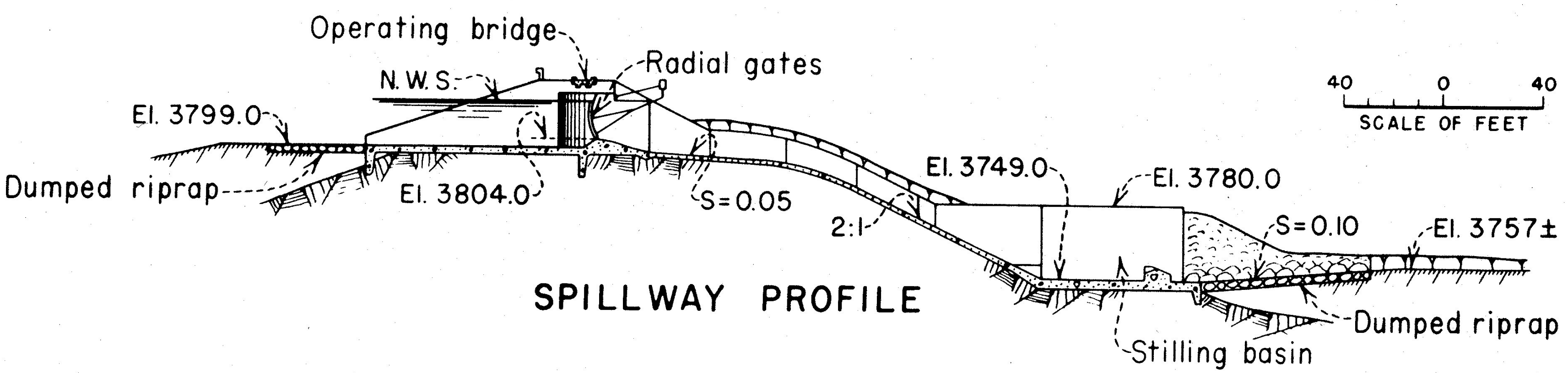
Hydraulic structure that passes normal (operational) and/or flood flows in a manner that protects the structural integrity of the dam and/or dikes.



Emergency Spillway

- Emergency spillway provides additional protection against dam and/or dike overtopping and intended for use under unusual or extreme conditions.

Spillway Profile



Spillway Walls

Stampede Dam Control Structure

Pier

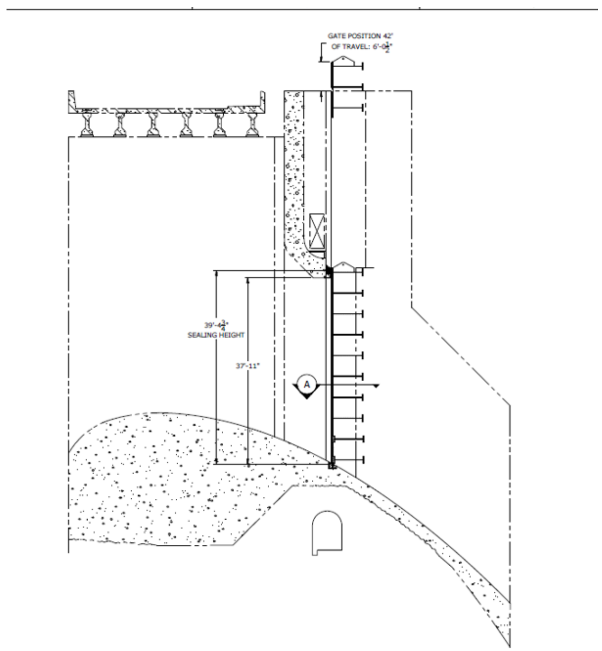
Canyon Ferry Dam Gate Piers

Glen Canyon Dam Gate Piers

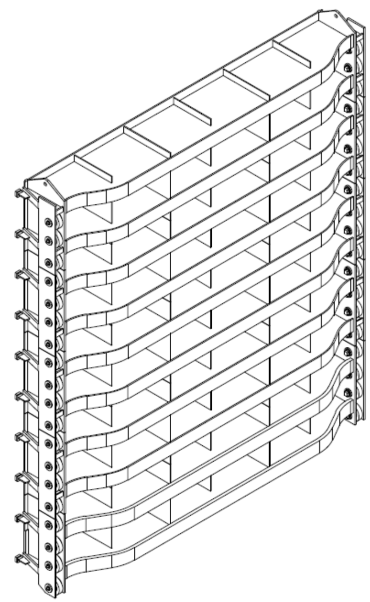
Minidoka Dam Canal Headworks Gate Piers

Figure E-2-4.—Typical spillway pier configurations.

Gate



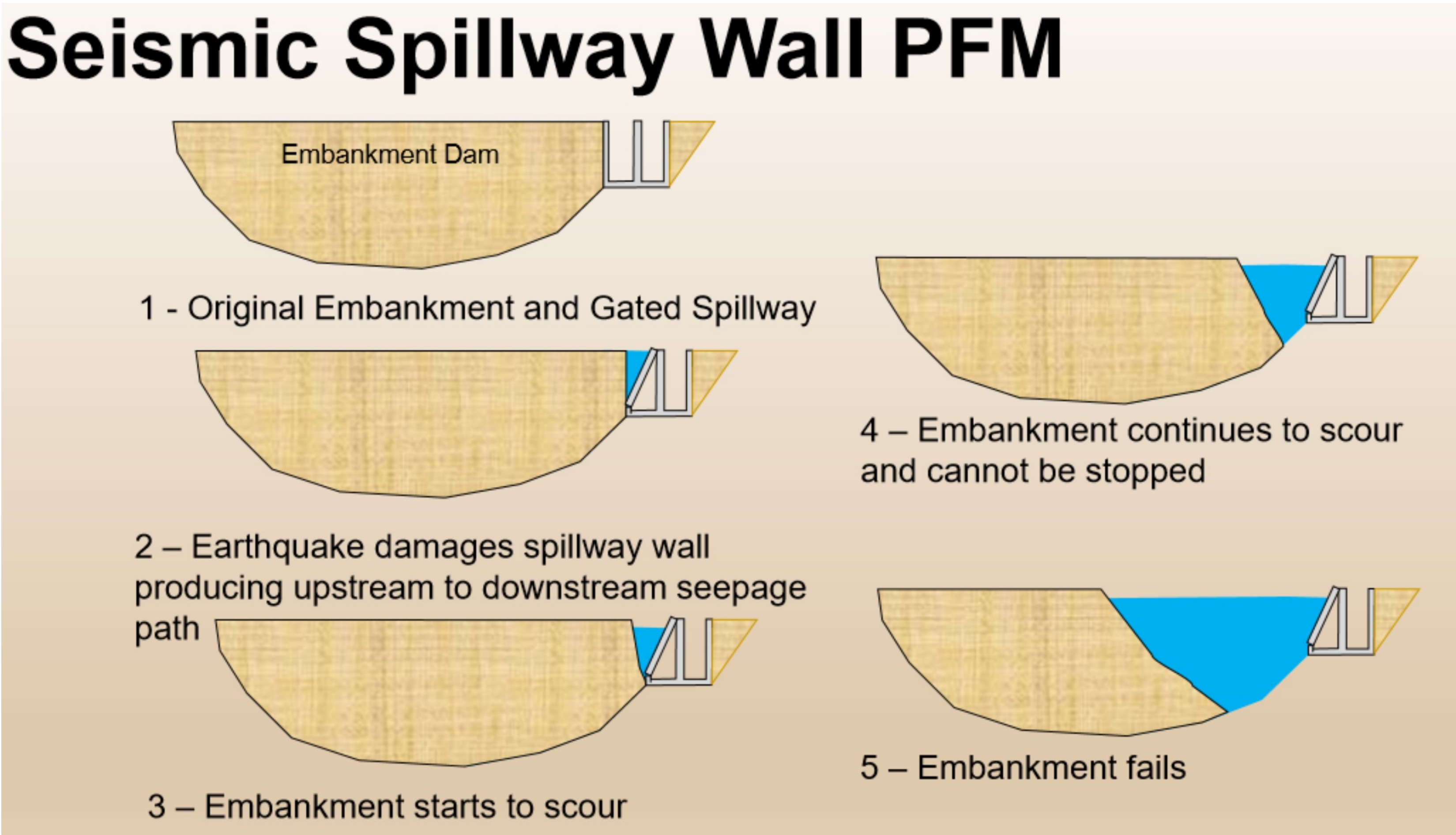
SPILLWAY GATE SECTION



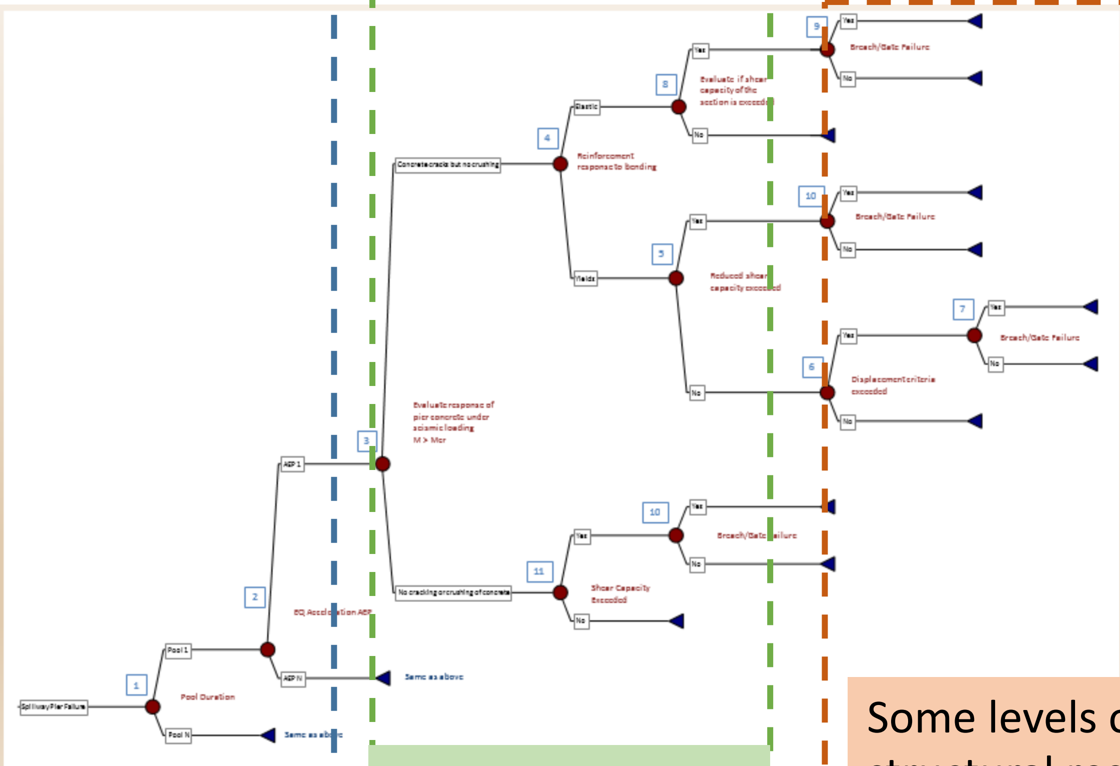
ISOMETRIC GATE LEAF

FEMA

Spillway Wall PFM discussion in Best Practice



Pier Discussion and its PFMs in Best Practice

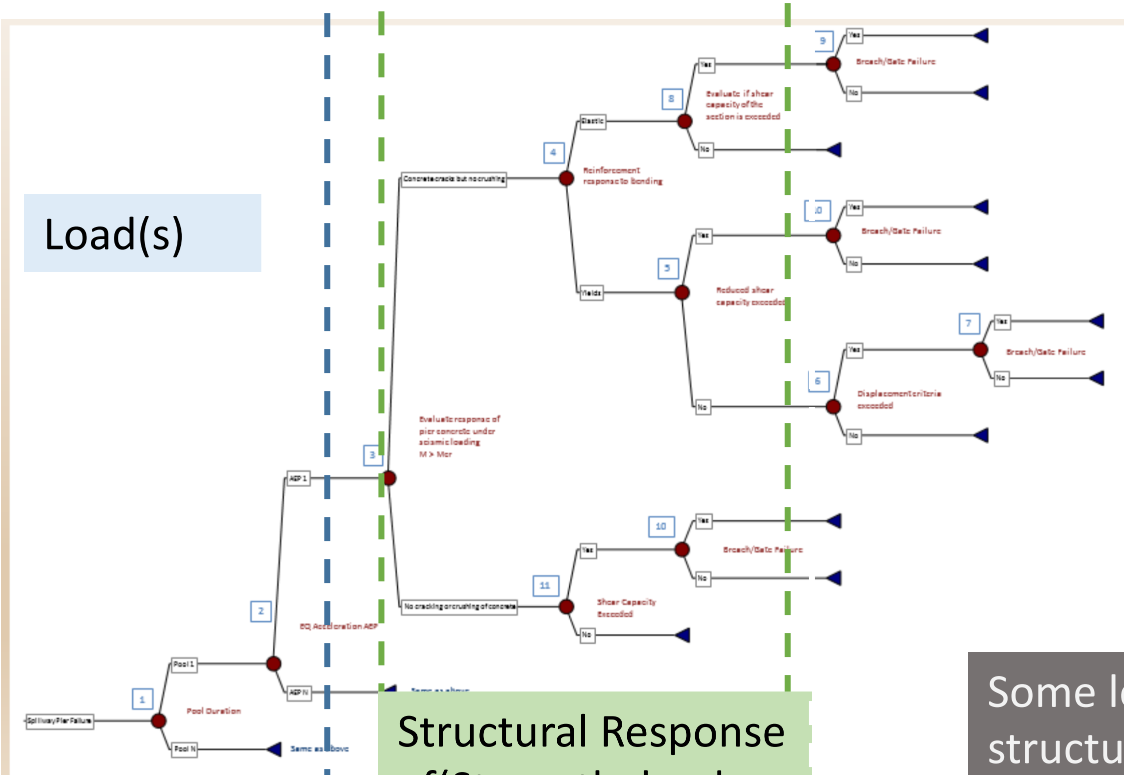


Load(s)

Structural Response =f(Strength, load path ability)

Some levels of warning from structural responses, while does not show here, the risk tree includes **intervention**=Impact of Human factor

Pier Discussion and its PFMs in Best Practice (2)



Load(s)

Structural Response =f(Strength, load path ability)

Some levels of warning from structural responses, while does not show here, the risk tree includes **intervention**=Impact of Human factor

Loading

To determine the appropriate seismic loads for a spillway, identification and evaluation of seismic-induced credible PFMs are undertaken (for more details, see Appendix B, "Potential Failure Modes (PFMs) for Spillways," in this chapter). If there are seismic-induced credible PFMs, the design load is determined through the process outlined in table 3.3.2-1. This process begins with assuming initial design loading conditions.

Analytical tools used to estimate the response of the structure to the earthquake loads involve pseudo-static and dynamic methods. These include:

- **Pseudo-static methods. –** These methods are typically used during appraisal and feasibility design. On occasion, these methods may be used during final design when dealing with common, simple structures without complex soil-structure interactions and that are subject to small to moderate seismic loading. These pseudo-static methods include:
  - **Westergaard method. –** The Westergaard method estimates hydrodynamic loading. For more details about applying the Westergaard method, see Chapter 6, "Structural Design Considerations for Spillways and Outlet Works," in this design standard.
  - **Mononobe-Okabe method. –** The Mononobe-Okabe (M-O) method estimates dynamic lateral soil loading. The M-O method computes the net static and dynamic force acting on a flexible (yielding) structure. For positive horizontal accelerations (soil accelerates toward the wall), the net dynamic active force (P_{AE}) is greater than the net static active force (P_a), and the net dynamic passive force (P_{PE}) is less than the net static passive force (P_p). Thus, compared with static conditions, the seismic earth pressures increase from the driving side soil mass and decrease from the resisting side soil mass. A limitation of the M-O method in higher seismic regions is that the soil angle of internal friction (φ) [47] must be greater than the seismic inertial angle (ψ) [48], which is a function of the horizontal acceleration. The M-O equations yield negative radicals (complex numbers) under such large seismic accelerations. A summary of the fundamental M-O assumptions is presented below:

- The wall yields sufficiently when subjected to active pressures.
- The backfill is cohesionless.
- The soil is assumed to satisfy the Mohr-Coulomb failure criterion.
- When the minimum active pressure is attained, a soil wedge behind the wall is at the point of incipient failure, and the maximum shear strength is mobilized along the potential slip plane.
- Failure in the backfill occurs along a slip plane surface that is inclined at some angle with respect to the horizontal backfill passing through the toe of the wall.
- The soil wedge behaves as a rigid body, and accelerations are constant throughout the mass.
- Equivalent static horizontal and vertical forces are applied at the center of gravity of the wedge and represent the earthquake forces.
- Liquefaction is not a consideration for the backfill.
- The backfill is completely above or completely below the water table, unless the ground surface is horizontal, in which case the backfill can be partially saturated.
- The ground surface is planar, not irregular or broken.
- Any surcharge is uniform and covers the entire soil surface.
- The soil angle of internal friction must be greater than the seismic inertial angle ($\varphi \geq \psi$).

For more details about applying the M-O method, see Chapter 23, "Seismic Failure of Spillway/Retaining Walls," of the *Dam Safety Risk Analysis Best Practices Training Manual* [6] and Chapter 6, "Structural Design Considerations for Spillways and Outlet Works," in this design standard.

- **Woods method. –** The Woods method estimates dynamic lateral soil loading (only applicable for nonyielding wall conditions). Woods method is based on linear elastic theory and on idealized representations of the wall-soil structural system. Elastic methods were originally developed and applied for the design of basement walls that would be expected to experience very small displacements

under seismic loading and, as such, can be considered as rigid, nonyielding walls. The fundamental assumption for the elastic methods is that the relative soil-structure displacement generates soil stresses in the elastic range of the material. Elastic methods are usually based on elastic wave solutions and are thought to represent upper-bound dynamic earth pressures and, as a result, produce seismic loads greater than those of the M-O method. Wood's method predicts a total dynamic thrust acting at a height equal to approximately 0.58H above the base of the wall. A summary of the fundamental Woods assumptions is presented below:

- The wall is a rigid, non-yielding wall.
- Soil stresses are in the elastic range.
- Computed dynamic thrust loads must be added to static lateral earth loads.
- Computed dynamic thrust loads are a function of the soil Poisson's ratio.
- Computed dynamic thrust loads are a function of the ratio of the effective horizontal length of the backfill to the height of the backfill.
- Not limited for large seismic accelerations.
- The earthquake shaking frequency is much less than the fundamental frequency of the backfill.

For more details about applying Woods method, see Chapter 23, "Seismic Failure of Spillway/Retaining Walls," of the *Dam Safety Risk Analysis Best Practices Training Manual* [6] and Chapter 6, "Structural Design Considerations for Spillways and Outlet Works," in this design standard.

- **Self-weight inertia (added mass). –** Any pseudo-static analysis will include the inertia forces associated with earthquake-induced acceleration of the spillway structure or feature, such as a wall. For more details, see Chapter 6, "Structural Design Considerations for Spillways and Outlet Works," in this design standard.

Dynamic methods. – Linear and nonlinear two-dimensional and three-dimensional Finite Element Model (FEM) methods are typically employed for some feasibility designs and for some final design level efforts (not all high-level designs will require FEM methods). Also, these methods are

Loading, design criteria for concrete retaining walls-1971 publication

DESIGN CRITERIA FOR
CONCRETE RETAINING WALLS

Report of the task committee on
design criteria for retaining walls

A. J. Aisenbrey, Jr.
R. B. Campbell
R. W. Kramer
J. Legas
L. M. Stimson

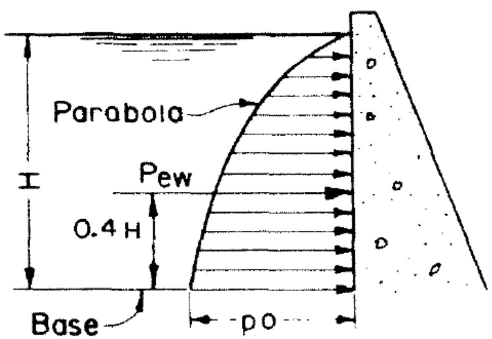
Division of Design
Engineering and Research Center
Denver, Colorado

First printed August 1971
Revised and Reprinted July 1977

UNITED STATES DEPARTMENT OF THE INTERIOR * BUREAU OF RECLAMATION

Loading (2)

Seismic water pressure on rigid wall



$$p_o = C_e \alpha_h H$$
$$P_{ew} = \frac{2}{3} C_e \alpha_h H^2$$
$$M = \frac{4}{15} C_e \alpha_h H^3$$

α_h = horizontal ground acceleration/g

T = earthquake period (sec)

$$C_e = \frac{51}{\sqrt{1-0.72\left(\frac{H}{1000T}\right)^2}}$$

See Reference 22.

Seismic water pressure on flexible wall

2. From Equation 15 and Figure 8, 5 percent damping,[5] determines whether a magnification factor, MF, need be applied. If MF does not apply, use Westergaard's parabola, the same as though the walls were rigid.

3. If magnification is required, the variation is assumed to be linear as shown by Line DE, Figure 11.[11] The curve AKC represents Westergaard's parabola as modified by magnification. The curve is produced by assuming values of Y and solving for X.

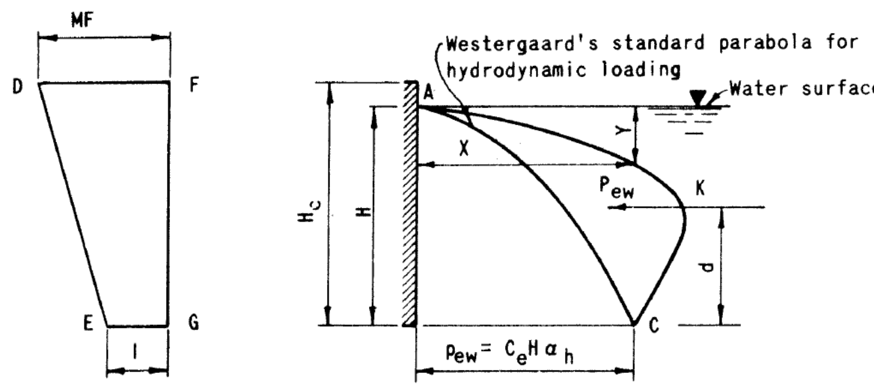


Figure 11. - Seismic water pressure on flexible wall.

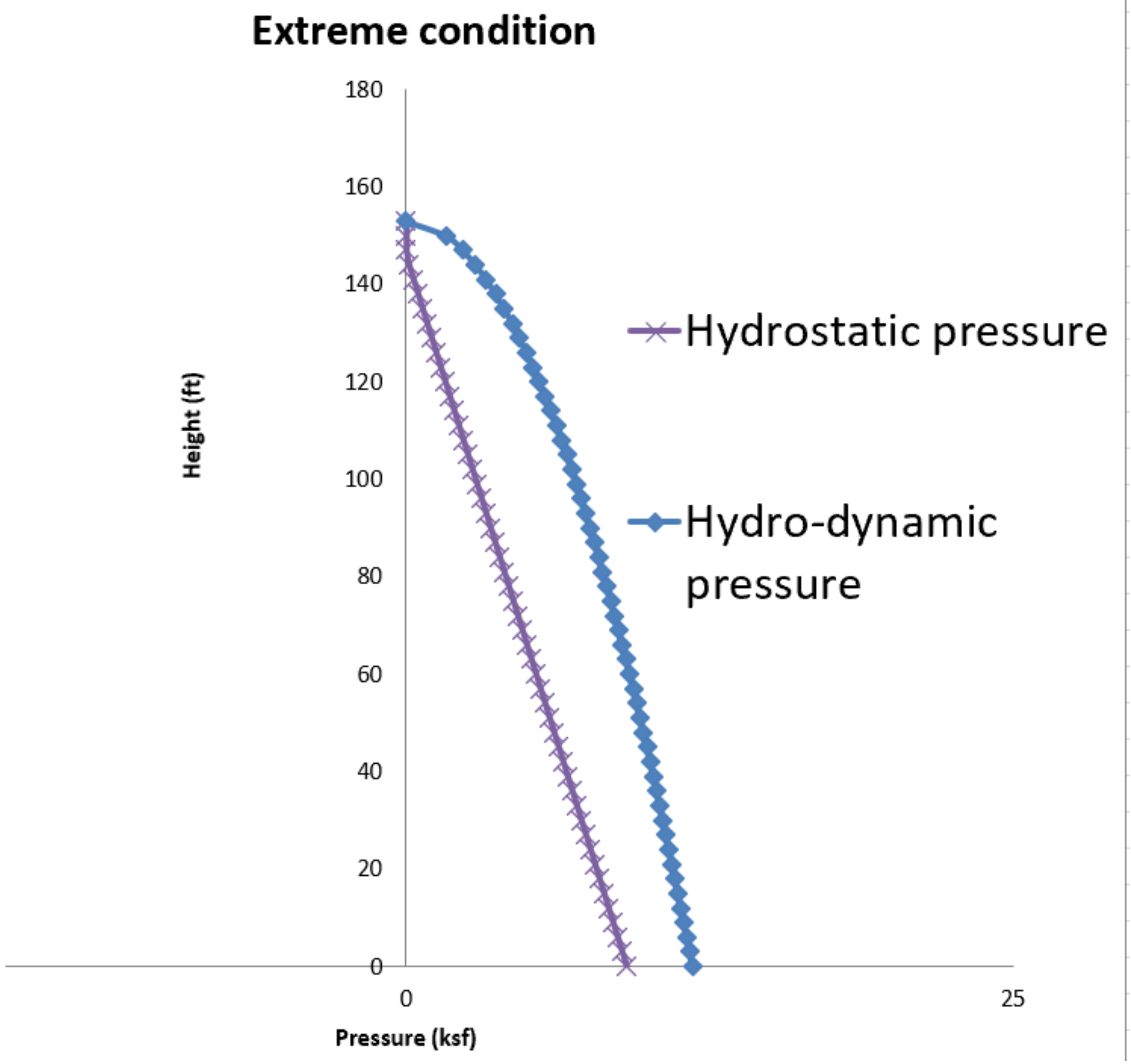
Then

$$X = C_e \alpha_h \sqrt{HY} \left[MF - \left(\frac{MF - 1}{H} \right) Y \right] . \qquad (16)$$

When Y = H, the total hydrodynamic load is:

$$P_{ew} = \frac{2}{3} C_e H^2 \ (0.4\ MF + 0.6)\ \alpha_h \qquad (17)$$

Loading



FEMA

Loading (3)

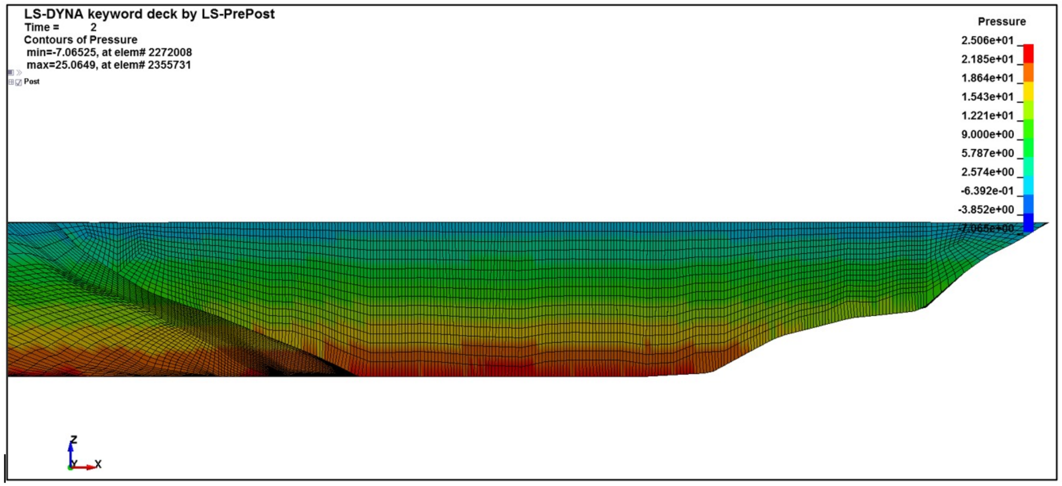


Figure 36. Contour plot shows water pressure in reservoir

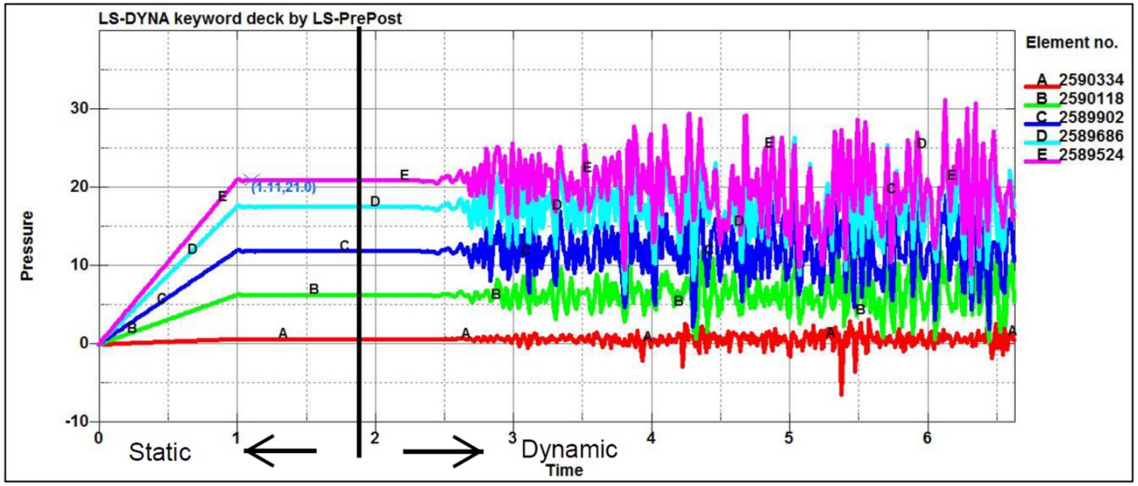
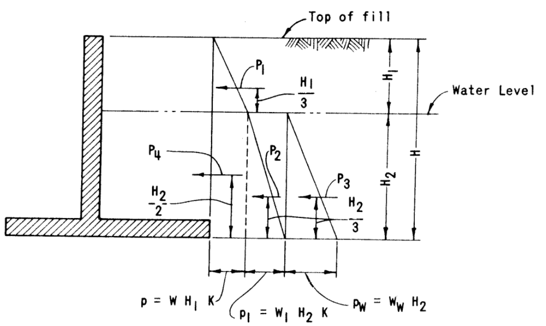


Figure 37. Data plot showing water pressure in the reservoir.

Pressure in soil, static and dynamic approximation

Static



W = drained weight of fill, lbs/cf; (fill above water level)

W_1 = buoyant weight of fill, lbs/cf; (fill below water level)

W_W = unit weight of water, lbs/cf = 62.4 lbs/cf

$W_1 = W_S - W_W$, where W_S is the unit weight of 100 percent saturated fill and can be determined from the relationship:

W_S = (oven-dry unit weight) + (62.4 x volume of voids)

$K = K_a$, K_p, or K_o (Rankine, Coulomb, or Jáky)

$$P_1 = W \frac{H_1^2}{2} K$$

$$P_2 = W_1 \frac{H_2^2}{2} K$$

$$P_3 = W_W \frac{H_2^2}{2}$$

$$P_4 = W H_1 K H_2$$

Dynamic

DYNAMIC LOADS

Seismic Fill Pressure

The total active fill force, P_{AE}, during an earthquake is obtained by adding a dynamic force component, ΔP_{AE}, to the active static force, P, described under Static Loads.[15]

$$P_{AE} = P + \Delta P_{AE} \qquad (7)$$

The components of P_{AE} are computed separately, since P acts at one-third the height of the fill above the base and ΔP_{AE} acts at two-thirds the height of the fill above the base.[9] [20] The force components are:

$$P = \frac{wH^2}{2} K_a \text{ and } \Delta P_{AE} = \frac{wH^2}{2} \Delta K_{AE}$$

then:

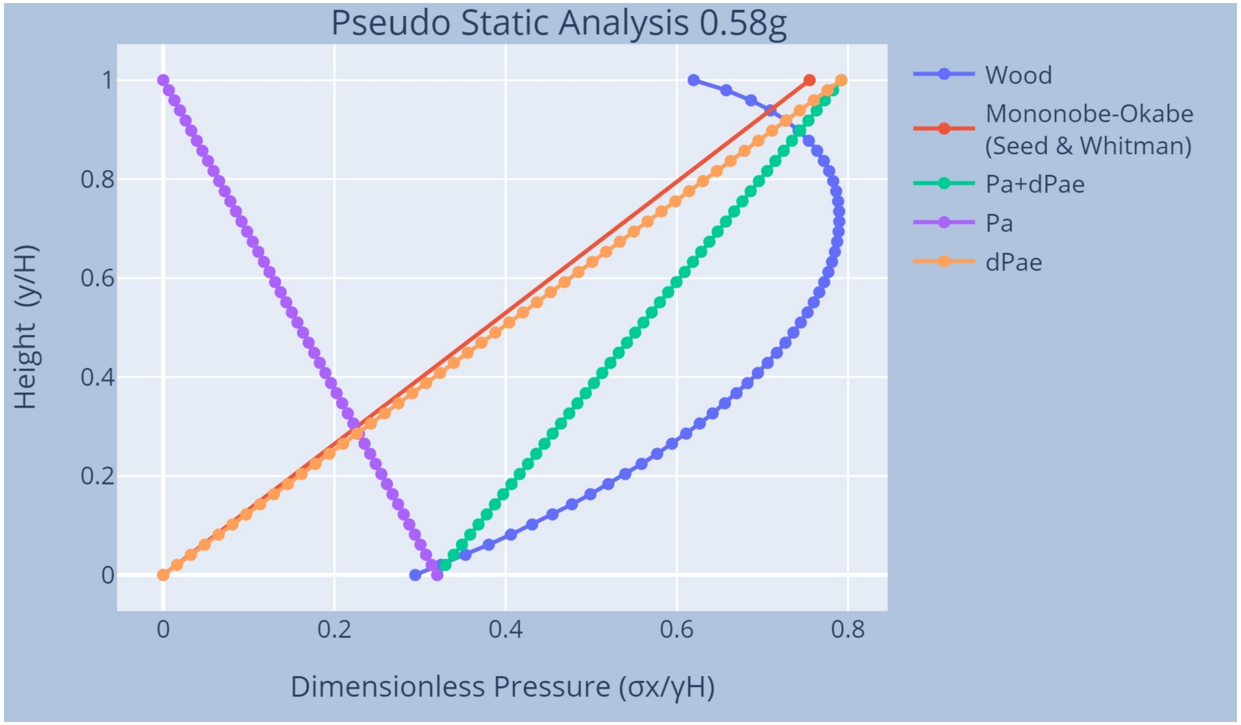
$$P_{AE} = \frac{wH^2}{2} (K_a + \Delta K_{AE}) = \frac{wH^2}{2} K_{AE} \qquad (8)$$

where:

ΔK_{AE} = dynamic increment of active earth pressure coefficient

K_{AE} = total active pressure coefficient

Pressure in soil, static and dynamic approximation (2)



Loadings and spillway structural evaluation prior to risk analysis

- Start with Linear elastic analysis
- Utilize the available load approximation methods
- Evaluate structural performance
- Utilize the approximation/relationship of structural performance and probability of failure
- Consider nonlinear analysis if needed, which allows team to review structural failure mechanisms (concrete cracks, steel yields)

Loadings and spillway structural evaluation prior to RA

site?
To what elevation of my structural the load reaches to?

Utilize the available load approximation methods

Evaluate structural performance

Utilize the approximation/relationship of structural performance and probability of failure

Consider nonlinear analysis if needed, which allows team to review structural failure mechanisms (concrete cracks, steel yields)

Evaluation in pier

Table 60.—2020 CR Pier Analysis Data and Strength Calculation

Property	Value	Unit	Reference
Concrete compressive strength, f'_c	4.13	ksi	See ECDA Section.
Computed concrete modulus of rupture, f_r	0.5	ksi	AASHTO Article 5.6.3.3
Concrete splitting test value, $f_{t\text{-}split}$	0.200	ksi	See ECDA Section, Lab recommendation
Recommended dynamic tensile, $f_{t\text{-}dynamic}$	0.290	ksi	See ECDA Section, Lab recommendation
Pier Geometry	**Value**	**Unit**	**Reference**
Cross canyon effective width, d	37	in	Dwg 258-D-180, net section at gate (Pier 3 and pier 4)
Stream direction base length, b	300	in	Dwg 258-D-180 (pier 4, section CC)
$I_g=bd^3/12$	1,235,561	in^4	
$y_t=0.5d$	18.3	in	
Estimated crack moment, $M_{cr}=f_r Ig/yt$	13,467	kip-in/section	AASHTO Equation 5.6.3.5.2-2, ACI Eqtn 24.2.3.4
Estimated crack moment, M_{cr}	44,891	lb-in/in	
Allowable moment, $M_a=(2/3)*M_{cr}$	29,927	lb-in/in	ACI 318-19 Equation 24.2.3.5
Steel area in pier per ft, A_s/ft	0.31	in^2/ft	
Steel yield strength, f_y	33,000	psi	CRSI, Engineering Data Report No. 48
Whitney stress block depth, a	2.91	in	
Bending Arm, d-0.5a	35.2	in	
Nominal flexural strength in pier, $Mn=A_s f_y(d-0.5a)$	**30043**	**lb-in/in**	
Beam thickness per foot, bw	12	in	
Beam effective depth, d	37	in	Dwg 258-D-180, net section at gate (Pier 3 and pier 4)
Steel ratio ρw=As/(bwd)	0.0007	unitless	
Normal concrete factor, λ	1		
Size effect modification factor, λs=(2/(1+0.1*d))0.5	0.65		ACI 318-19-22.5.5.1.3
shear stress, vc=8(λs)(λ)(ρw)(1/3)(f'c)0.5	30	psi	ACI 318-19-Table 22.5.5.1-c, with Nu=0
Shear, Vc=vc*d	**1098**	**lb/in**	
Reduced shear, 0.75*Vc	**824**	**lb/in**	

Table 61.—Pier Analysis Flexural Results for 10K and 50K Event

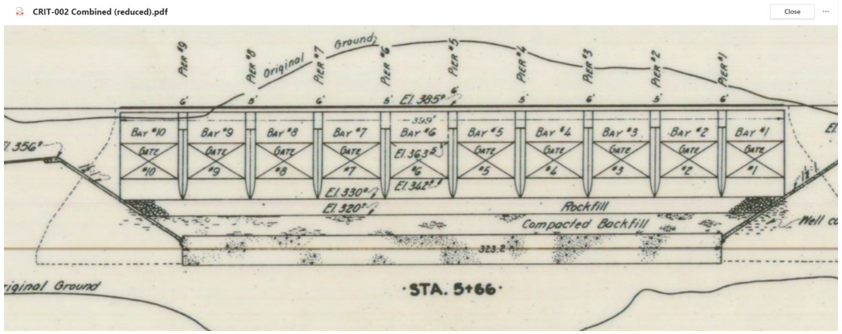
Analysis Results	10K event (0.27g)	50K event (0.58g)
Average bending moment in pier from model, M_u, lb-in/in	28,000	60,000
Allowable moment, M_a, lb-in/in	29,927	29,927
Nominal flexural strength in pier, M_n, lb-in/in	30,043	30,043
D/C ratio = M_u /M_a	0.9	2.0
Check	OK	No Good

Table 62.—Pier Analysis Shear Results for 10K and 50K Event

Analysis Results	10K event (0.27g)	50K event (0.58g)
Average shear in pier from model, lb/in	418	896
Reduced (allowable) shear in pier, V_n, lb/in	824	824
D/C ratio = Shear in model/Vn	0.5	1.1
Check	OK	No Good

FEMA

Linear analysis in pier



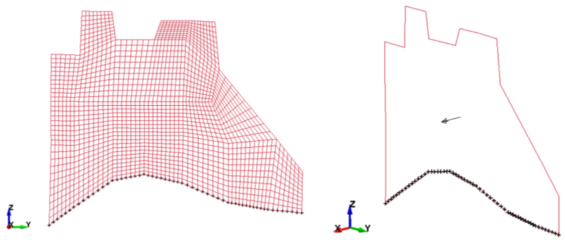


Figure 5.—Showing the pier model and boundary condition between pier base and spillway crest interface (left) and location of the earthquake load assigned to the model (right).

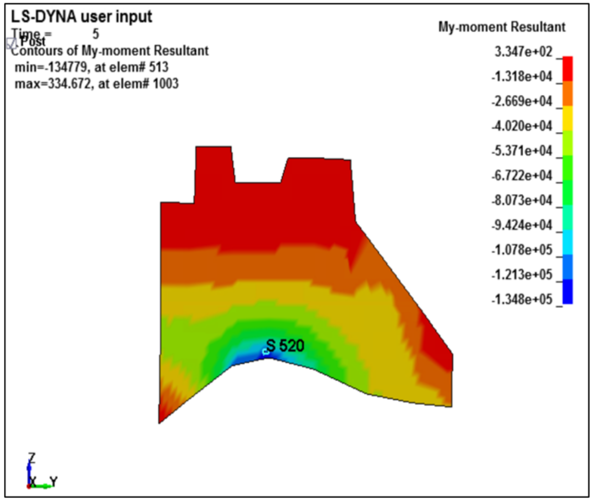


Figure 6. Showing model results (flexural stress contour) from 10k event

FEMA

Linear analysis in pier (2)

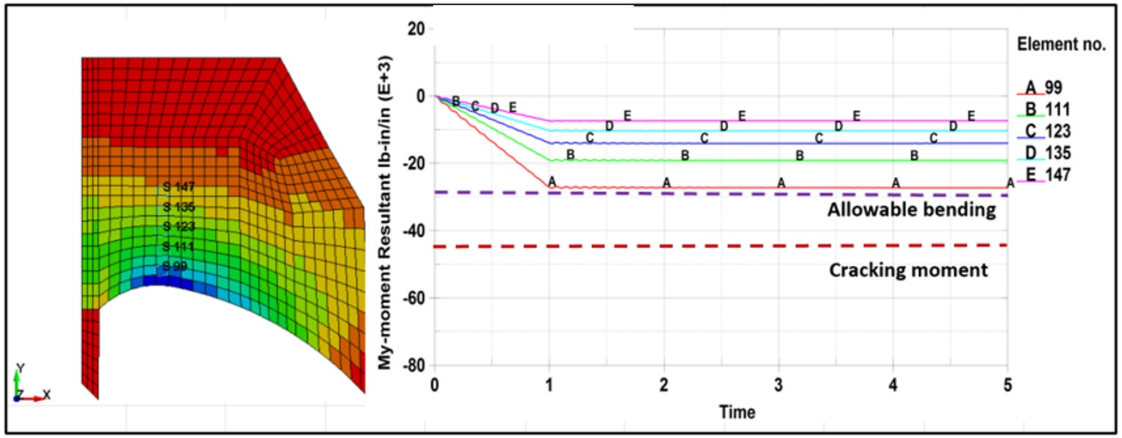


Figure 8.—Showing model results (bending moments) from 10k event and allowable bending moment in the pier per AASHTO code.

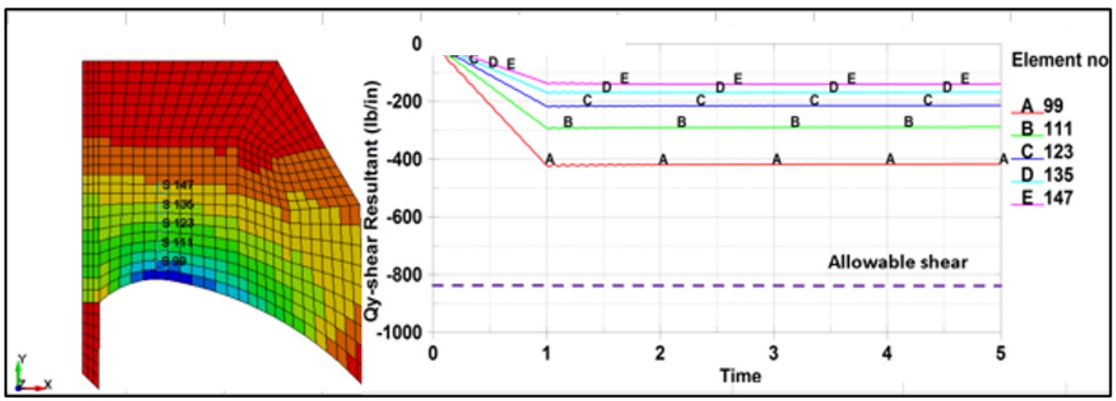
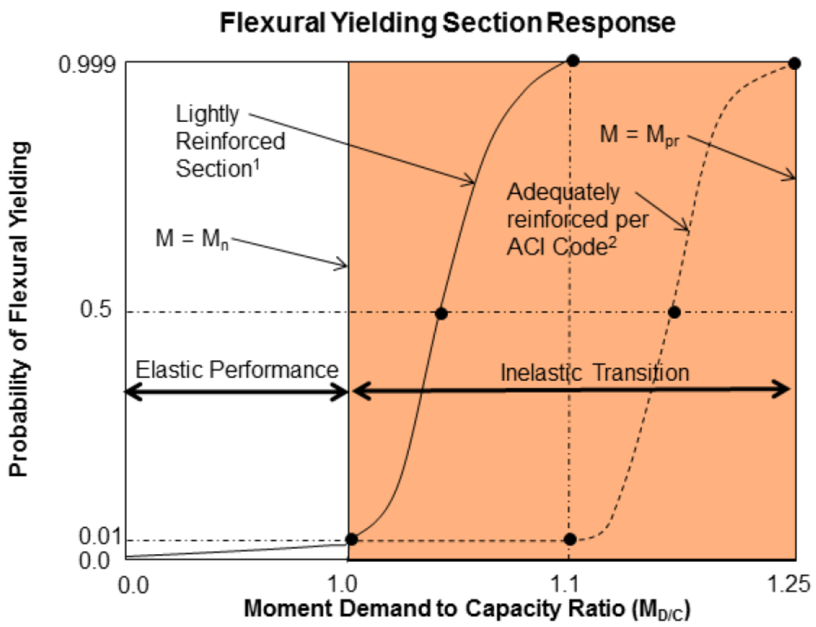


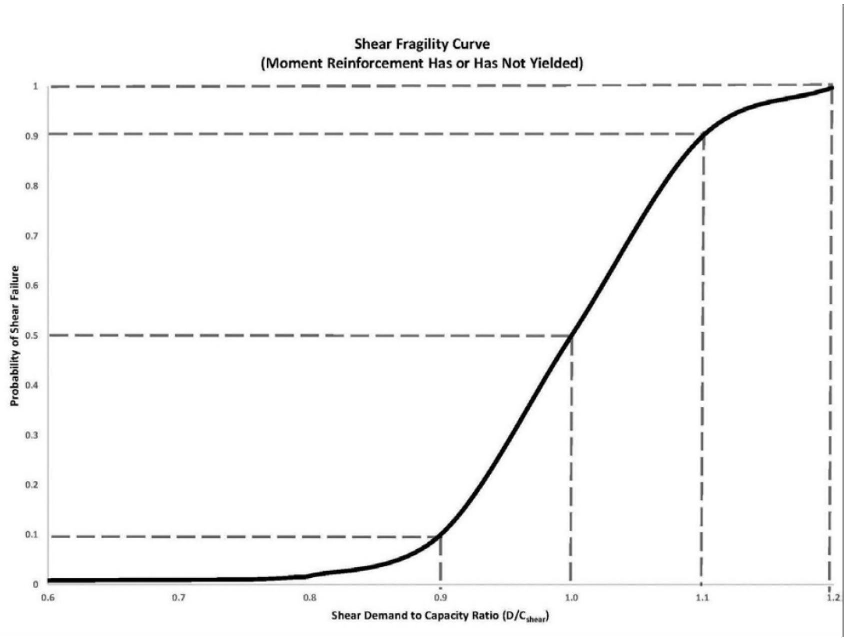
Figure 9.—Showing model results (shear) from 10k event and allowable bending moment in the pier per ACI318-19 code.

Loadings and spillway structural evaluation prior to RA (2)

- Start with Linear elastic analysis
- Utilize the available load approximation methods
- Evaluate structural performance
- Utilize the approximation/relationship of structural performance and probability of failure
- Consider nonlinear analysis if needed, which allows team to review structural failure mechanisms (concrete cracks, steel yields)

Best Practice references

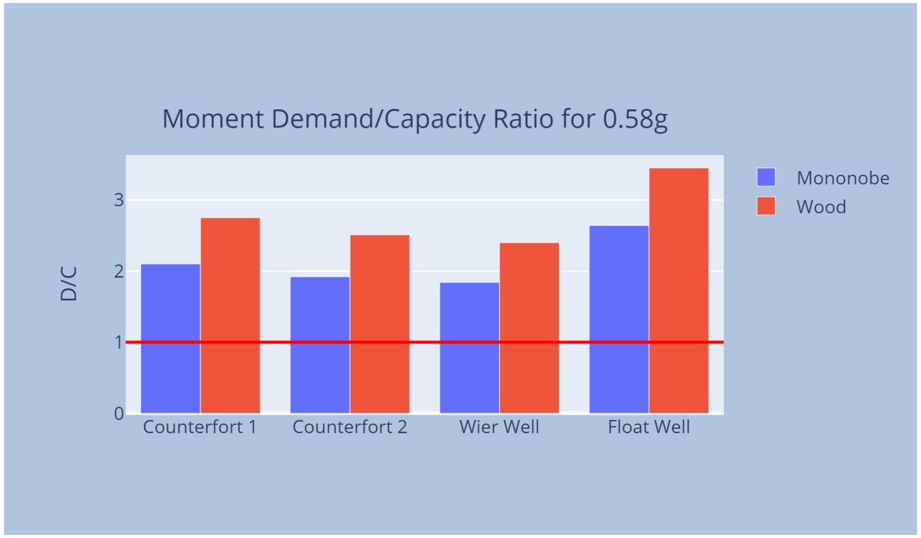


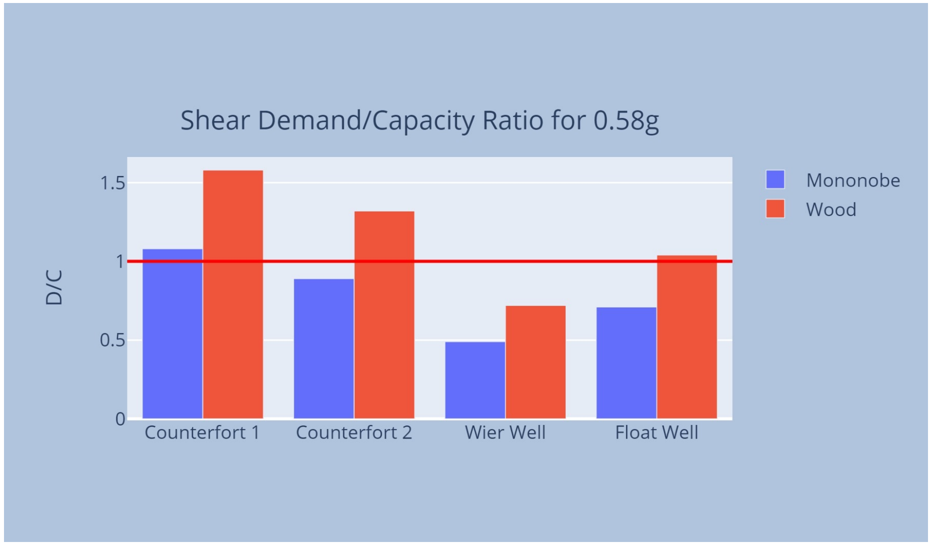


FEMA

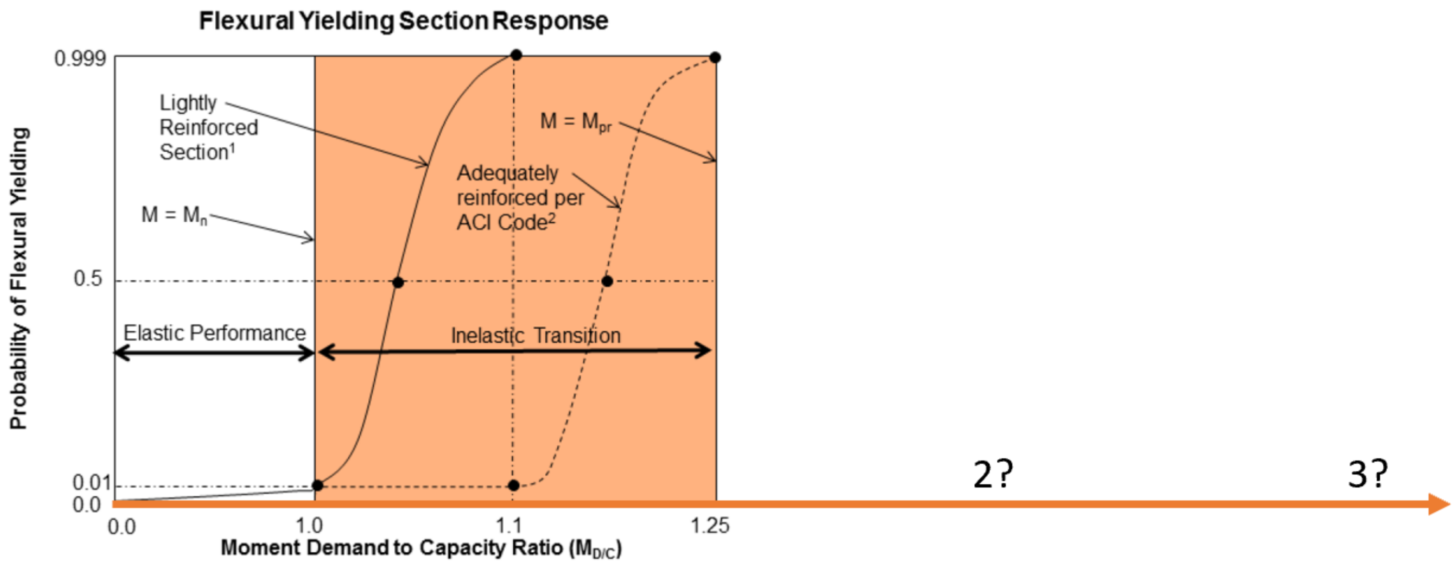
What if the structural responses show nonlinear results?

Signs of nonlinear





Sign of nonlinear



BOR Spillway inventory list 57 spillways that are designed before and in the 1940s.

The first seismic design code for buildings was published in 1940, one year after the destructive Erzincan earthquake.

EVOLUTION OF SEISMIC BUILDING DESIGN AND CONSTRUCTION PRACTICE IN TURKEY (https://pubs.usgs.gov/of/2001/of01-163/GENERAL_PUBLICATIONS/Sezen_StructDesignofTallBld.pdf)

Loadings and spillway structural evaluation prior to RA (3)

- Start with Linear elastic analysis
- Utilize the available load approximation methods
- Evaluate structural performance
- Utilize the approximation/relationship of structural performance and probability of failure
- Consider nonlinear analysis if needed, which allows team to review structural failure mechanisms (concrete cracks, steel yields)

FEMA

Nonlinear evaluation example

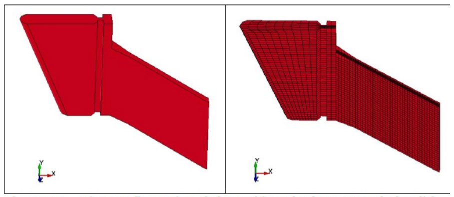


Figure 18.—Pier configuration (left) and hexahedron-(8 noded-solid element) mesh (right) constructed in the model.

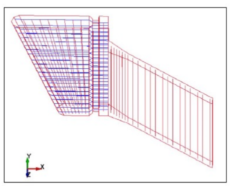


Figure 19.—Plots shows rebar mesh modeled inside the concrete.

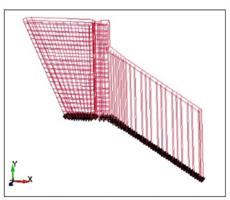


Figure 20.—Plot shows fixity (in both translation and rotation) boundary condition assigned at the base of the pier.

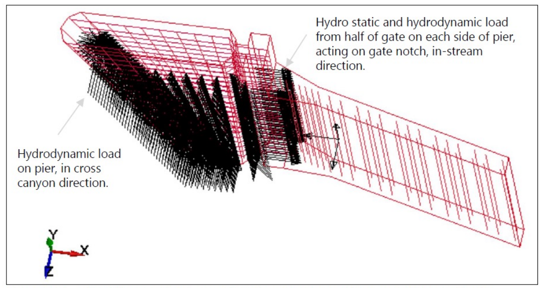


Figure 21.—Plot shows seismic acceleration at center of mass and water load application planes for both hydro static and hydro-dynamic load.

Figure 22 shows cracks in concrete and tensile rebars yielding (Figure 23) in the 50,000-year seismic event for Loading Combination 1: 100 percent seismic loading acting in the in-stream direction plus 30 percent seismic loading acting in cross canyon direction.

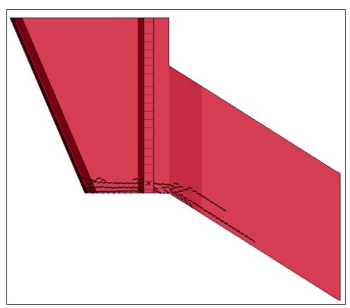


Figure 22.—Concrete cracks at end of pier in the 50,000-year event, load combination 1: (100 percent instream and 30 percent cross canyon).

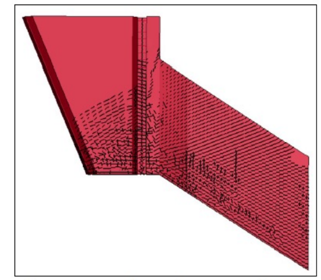


Figure 24.—Plots shows cracks registered in the concrete pier in the 50,000 yr-load combination 2: (30 percent instream+100 percent cross canyon).

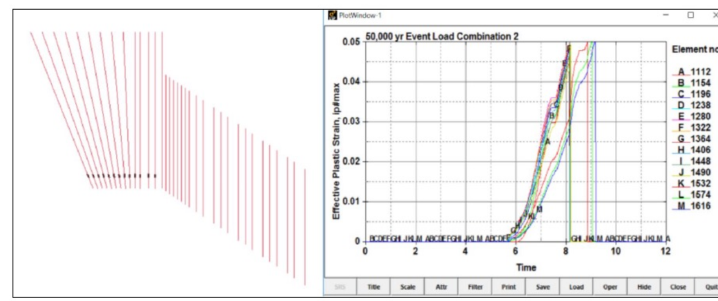


Figure 25.—Tensile rebars rupture in the 50,000 yr event, load combination 2: (30 percent instream and 100 percent cross canyon).

Nonlinear evaluation example (2)

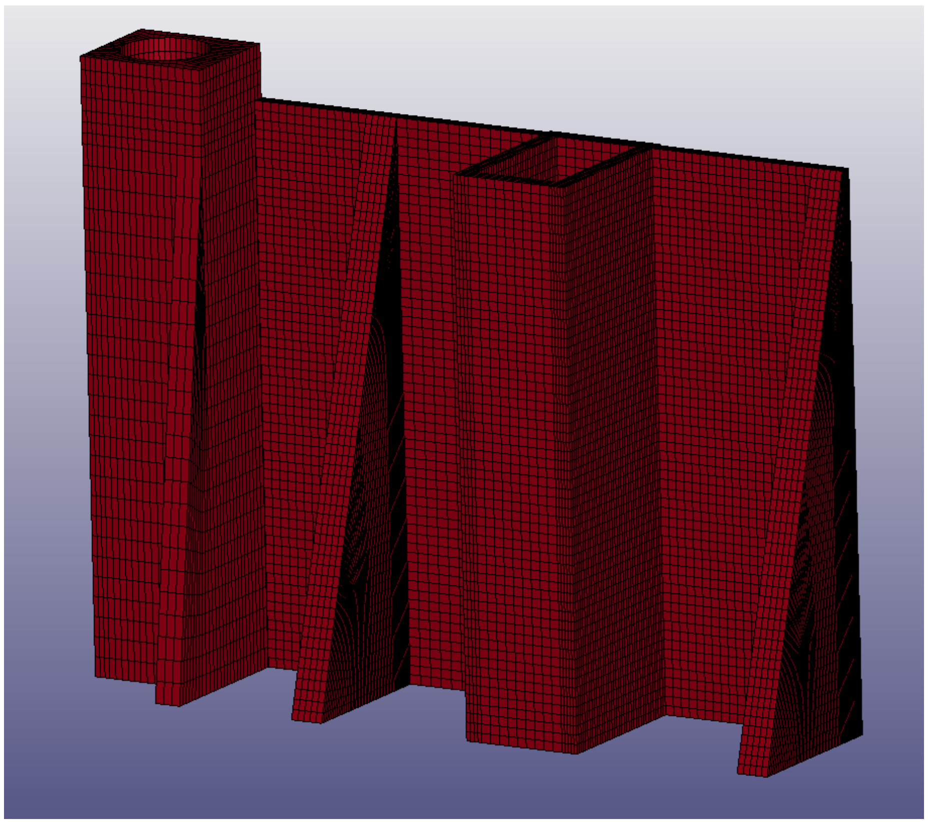
Load

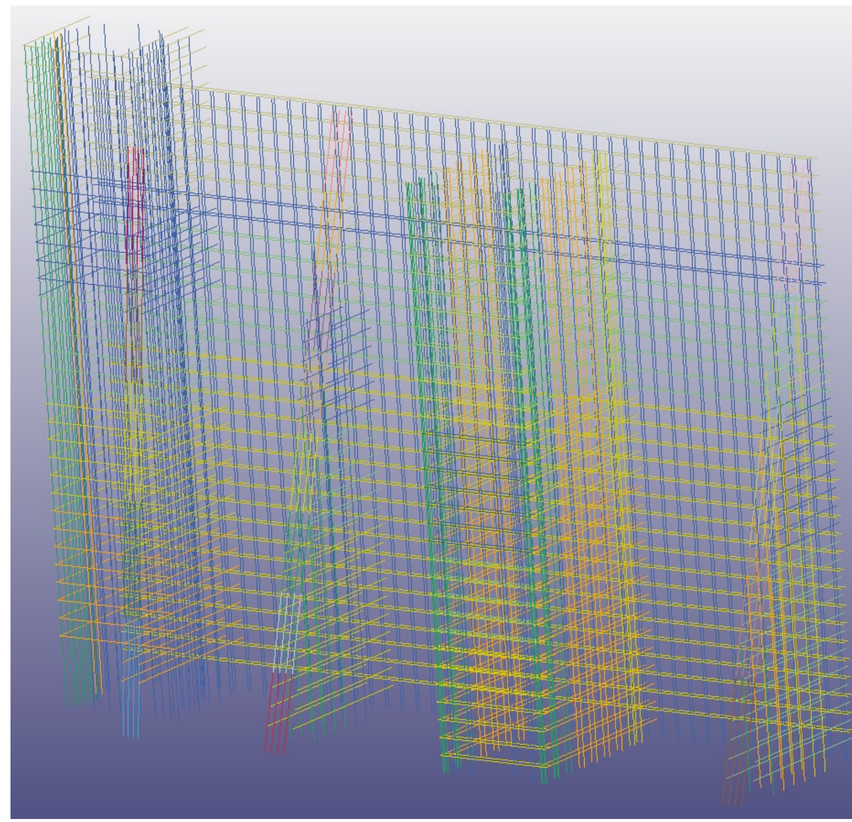
Structural responses, RC failure mechanism

Table 13.—Summary FEA results for spillway pier

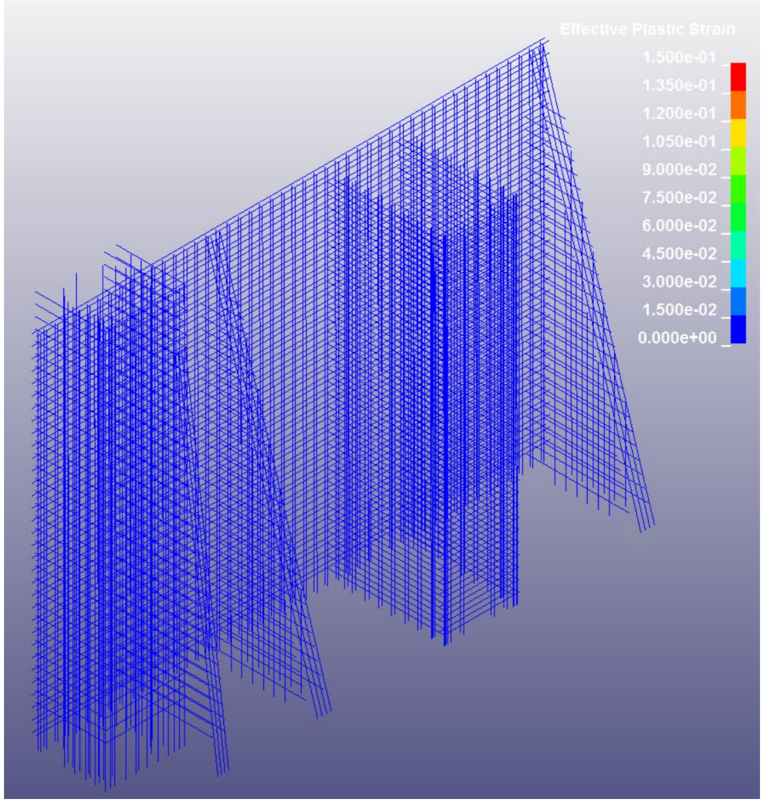
Seismic event	Loading combination	Shear demand/ capacity	Concrete crack?	Tensile rebars yield?	Tensile rebar ruptures?
50,000 years, PHA=1.04g	(1) 100% instream+30%cross canyon	1.0	Yes	Yes	No
	(2) 30% instream+100%cross canyon	1.0	Yes	Yes	Yes
10,000 years, PHA=0.4g	(1) 100% instream+30%cross canyon	0.5	Yes	No	No
	(2) 30% instream+100%cross canyon	0.5	Yes	Yes	No
5,000 years, PHA=0.25g	(1) 100% instream+30%cross canyon	0.4	Yes	No	No
	(2) 30% instream+100%cross canyon	0.4	Yes	No	No

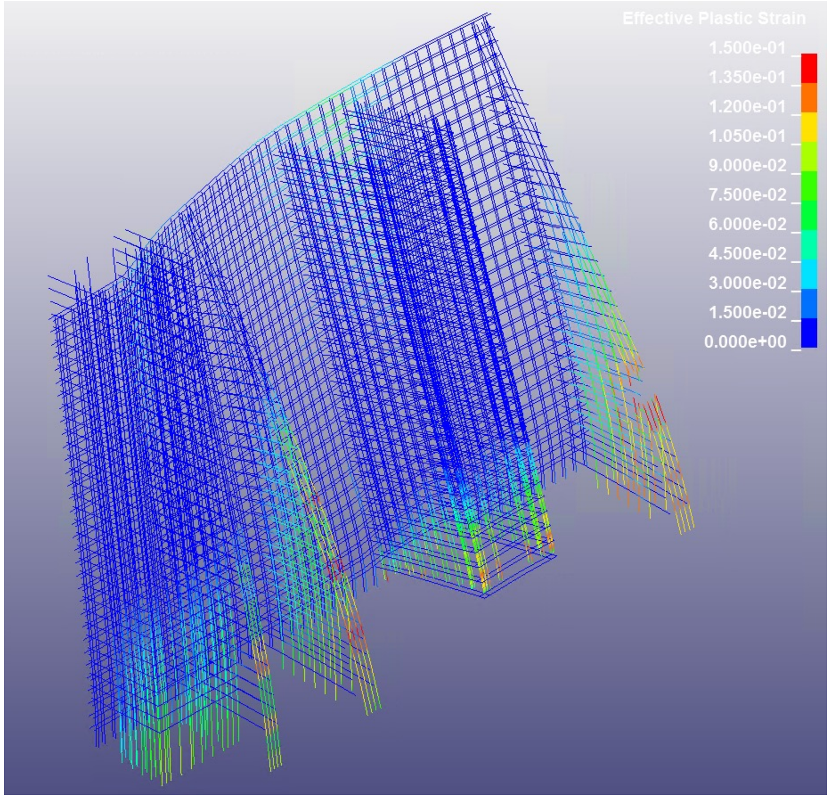
Wall nonlinear example/State of reinforcement





Wall nonlinear example/State of reinforcement (2)





Gate (2)

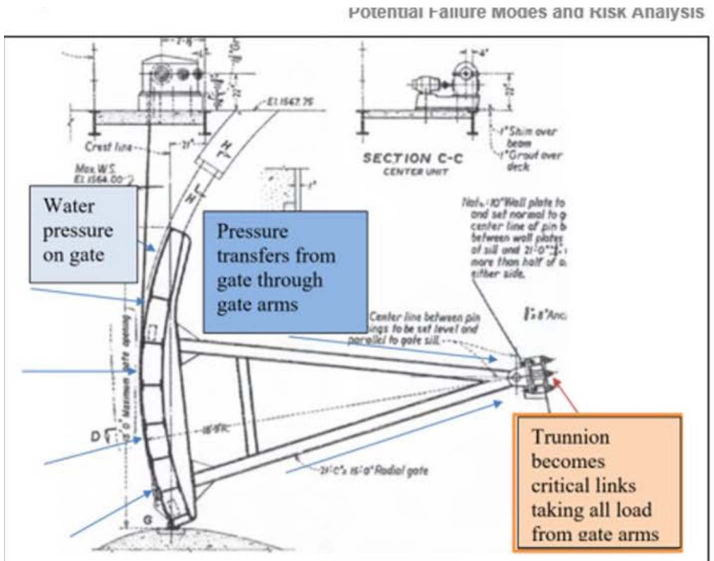
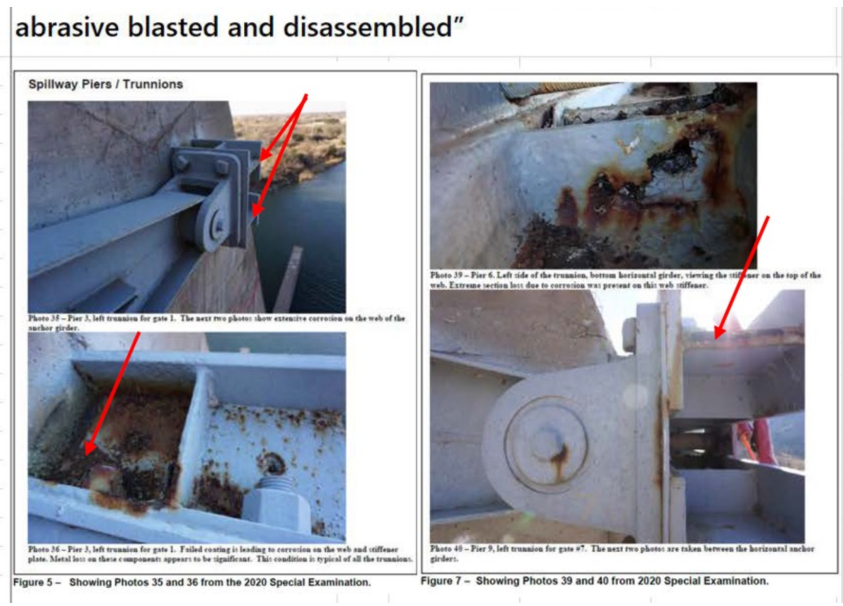


Figure 4 – Section through the spillway (Source: Dwg. 258-D-178).



abrasive blasted and disassembled"

Spillway Piers / Trunnions

Photo 35 – Pier 3, left trunnion for gate 1. The next two photos show extensive corrosion on the web of the anchor girder.

Photo 36 – Pier 3, left trunnion for gate 1. Failed coating is leading to corrosion on the web and stiffener plate. Metal loss on these components appears to be significant. This condition is typical of all the trunnions.

Figure 5 – Showing Photos 35 and 36 from the 2020 Special Examination.

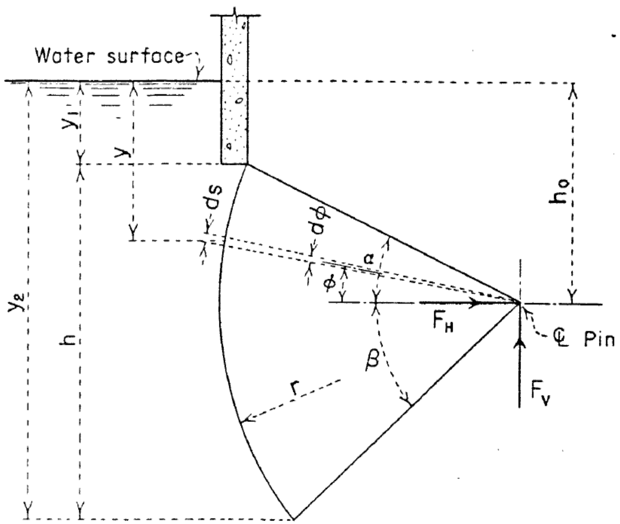
Photo 39 – Pier 6. Left side of the trunnion, bottom horizontal girder, viewing the stiffener on the top of the web. Extreme section loss due to corrosion was present on this web stiffener.

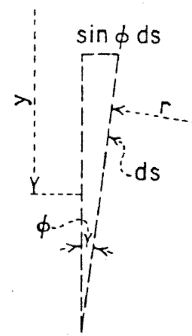
Photo 40 – Pier 9, left trunnion for gate #7. The next two photos are taken between the horizontal anchor girders.

Figure 7 – Showing Photos 39 and 40 from 2020 Special Examination.

Gate (3)

Load





$ds = r\,d\phi$

$y = h_0 + r\sin\phi$

w = width of gate

Net vertical force $= 62.4\, w \int_{-\alpha}^{\beta} y \sin\phi \, ds$

$$= 62.4\, wr \int_{-\alpha}^{\beta} (h_0 + r\sin\phi)\sin\phi\, d\phi$$

Let $62.4\, wr = k$

Then net vertical force $= k\left[-h_0\cos\phi + r\left(\frac{\phi\frac{\pi}{180}}{2} - \frac{\sin 2\phi}{4}\right)\right]_{-\alpha}^{\beta}$

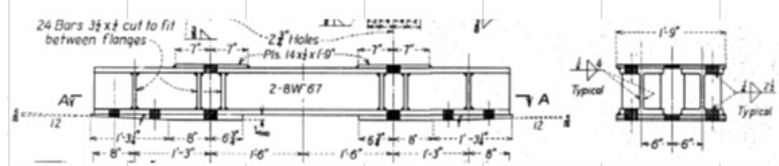
$$F_V = k\left[+h_0(\cos\alpha - \cos\beta) + \frac{\pi r}{360}(\alpha+\beta) - \frac{r}{4}(\sin 2\alpha + \sin 2\beta)\right]$$

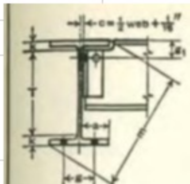
α is negative when $y_1 > h_0$ $\begin{cases}\cos\alpha \text{ is positive} \\ \sin\alpha \text{ is negative}\end{cases}$

β is negative when $h_0 > y_2$ $\begin{cases}\cos\beta \text{ is positive} \\ \sin\beta \text{ is negative}\end{cases}$

Strength

Sources:DWG 258-D-178, 258-D-180 and 258-D-221



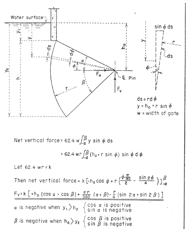


WF SHAPES

DIMENSIONS FOR DETAILING

Nominal Size	Weight per Foot	Depth	Flange Width	Flange Thickness	Web Thickness	Web Half Thickness	Distance a	Distance T	Distance k	Distance m	Distance g_1	Distance c	Usual Gage g
In.	Lb.	In.	In.	In.	In.	In.	In.	In.	In.	In.	In.	In.	In.
10 x 10	112	11 3/8	10 3/8	1 1/4	3/4	3/8	4 7/8	7 7/8	1 3/4	15 1/2	3	7/16	5 1/2
	100	11 1/8	10 3/8	1 1/8	11/16	3/8	4 7/8	7 7/8	1 5/8	15 1/4	3	7/16	5 1/2
	89	10 7/8	10 1/4	1	5/8	5/16	4 7/8	7 7/8	1 1/2	15	2 3/4	3/8	5 1/2
	77	10 5/8	10 1/4	7/8	9/16	5/16	4 7/8	7 7/8	1 3/8	14 3/4	2 3/4	3/8	5 1/2
	72	10 1/2	10 1/8	13/16	1/2	1/4	4 7/8	7 7/8	1 5/16	14 5/8	2 3/4	5/16	5 1/2
	66	10 3/8	10 1/8	3/4	7/16	1/4	4 7/8	7 7/8	1 1/4	14 1/2	2 1/2	5/16	5 1/2
	60	10 1/4	10 1/8	11/16	7/16	1/4	4 7/8	7 7/8	1 3/16	14 3/8	2 1/2	5/16	5 1/2
	54	10 1/8	10	5/8	3/8	3/16	4 7/8	7 7/8	1 1/8	14 1/4	2 1/2	1/4	5 1/2
	49	10	10	9/16	3/8	3/16	4 7/8	7 7/8	1 1/16	14 1/8	2 1/2	1/4	5 1/2
10 x 8	45	10 1/8	8	5/8	3/8	3/16	3 7/8	7 7/8	1 1/8	13	2 1/2	1/4	5 1/2
	39	10	8	1/2	5/16	3/16	3 7/8	7 7/8	1 1/16	12 7/8	2 1/2	1/4	5 1/2
	33	9 3/4	8	7/16	5/16	3/16	3 7/8	7 7/8	15/16	12 5/8	2 1/4	1/4	5 1/2
10 x 5 3/4	29	10 1/4	5 3/4	1/2	5/16	3/16	2 3/4	8 1/2	7/8	11 3/4	2 1/4	1/4	2 3/4
	25	10 1/8	5 3/4	7/16	1/4	1/8	2 3/4	8 1/2	13/16	11 5/8	2 1/4	3/16	2 3/4
	21	9 7/8	5 3/4	5/16	1/4	1/8	2 3/4	8 1/2	11/16	11 1/2	2	3/16	2 3/4
8 x 8	67	9	8 1/4	15/16	9/16	5/16	3 7/8	6 3/8	1 5/16	12 1/4	2 3/4	3/8	5 1/2

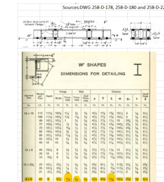
Gate (4)



Structural performance

Table 4.—D/C Ratio for Trunnion Web with respect to Each Combination

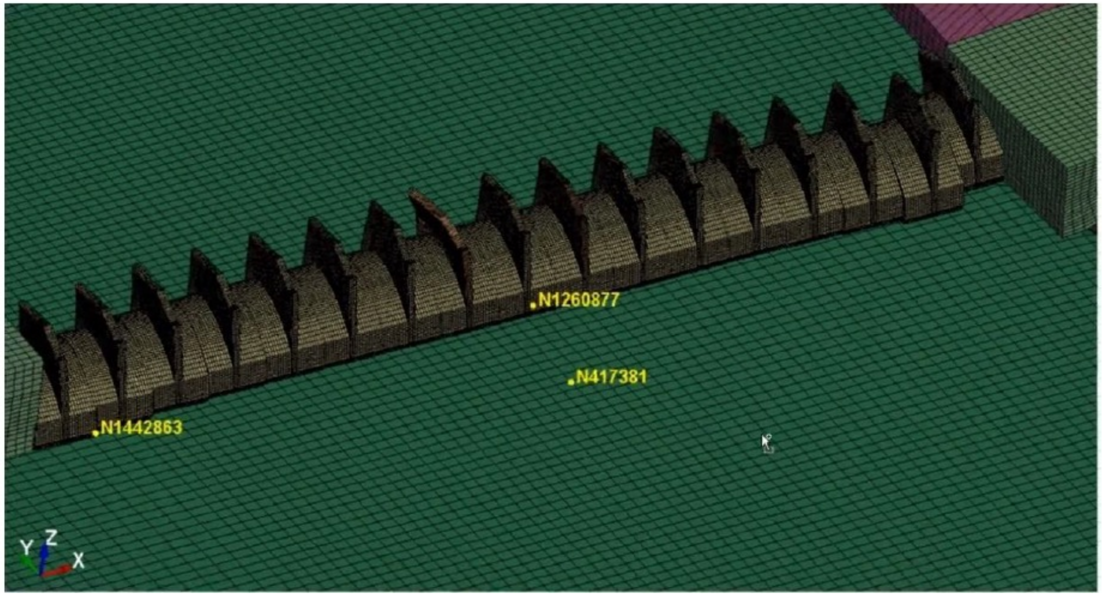
RWS Elev	Average 1	Average 2	Minimum 1+2	Average 1+2	Minimum 1+2+3+4	Average 1+2+3+4
	Total area of web, in^2=**2.394**	Total area of web, in^2=**2.950**	Total area of web, in^2=**4.058**	Total area of web, in^2=**5.344**	Total area of web, in^2=**7.217**	Total area of web, in^2=**9.611**
1547	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**
1548	**0.01**	**0.01**	**0.00**	**0.00**	**0.00**	**0.00**
1549	**0.03**	**0.03**	**0.02**	**0.01**	**0.01**	**0.01**
1550	**0.07**	**0.06**	**0.04**	**0.03**	**0.02**	**0.02**
1551	**0.13**	**0.10**	**0.07**	**0.05**	**0.04**	**0.03**
1552	**0.20**	**0.16**	**0.11**	**0.08**	**0.06**	**0.04**
1553	**0.28**	**0.23**	**0.15**	**0.11**	**0.09**	**0.06**
1554 (Top of current reservoir restriction elevation)	**0.39**	**0.31**	**0.21**	**0.16**	**0.12**	**0.09**
1555	**0.50**	**0.41**	**0.27**	**0.20**	**0.15**	**0.11**
1556	**0.63**	**0.51**	**0.34**	**0.26**	**0.19**	**0.14**
1557	**0.78**	**0.63**	**0.42**	**0.32**	**0.23**	**0.18**
1558	**0.94**	**0.76**	**0.50**	**0.38**	**0.28**	**0.21**
1559 (Normal Water Surface Elevation)	**1.12**	**0.91**	**0.60**	**0.45**	**0.34**	**0.25**
1560	**1.31**	**1.06**	**0.70**	**0.53**	**0.39**	**0.30**
1561	**1.52**	**1.23**	**0.81**	**0.61**	**0.45**	**0.34**
1562 (Top of gate)	**1.74**	**1.41**	**0.93**	**0.70**	**0.52**	**0.39**



Loadings and spillway evaluation prior to RA

- Start with Linear elastic analysis
- Utilize the available load approximation methods
- Evaluate structural performance
- Utilize the approximation/relationship of structural performance and probability of failure
- Consider nonlinear analysis if needed, which allows team to review structural failure mechanisms (concrete cracks, steel yields) or **even with 3D effect or other considerations**

3D effects



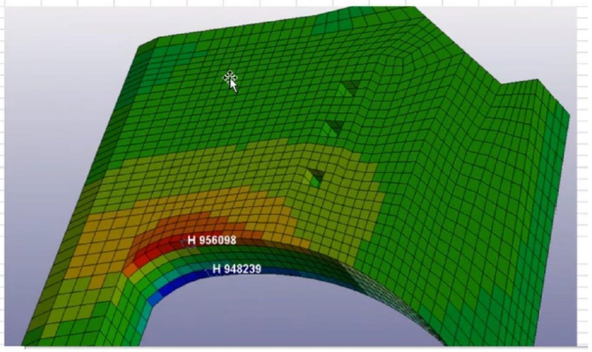
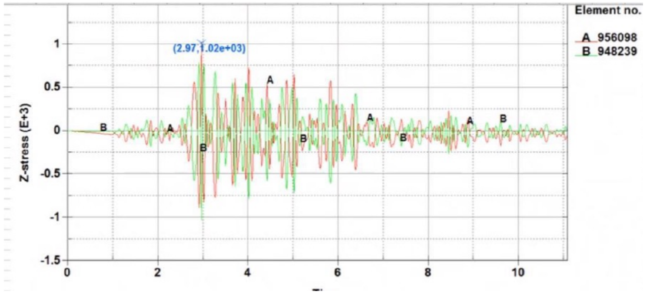


Figure 24.—Stress contour plot.



3D effects (2)

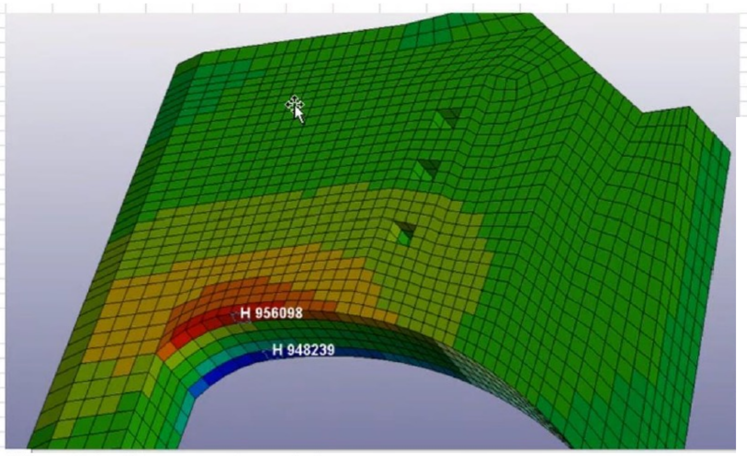
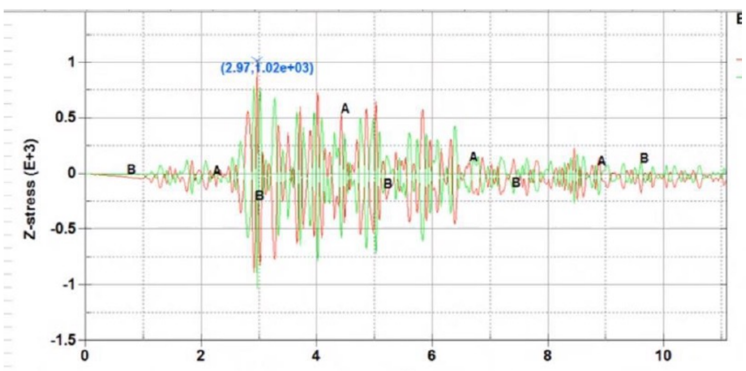


Figure 24.—Stress contour plot.



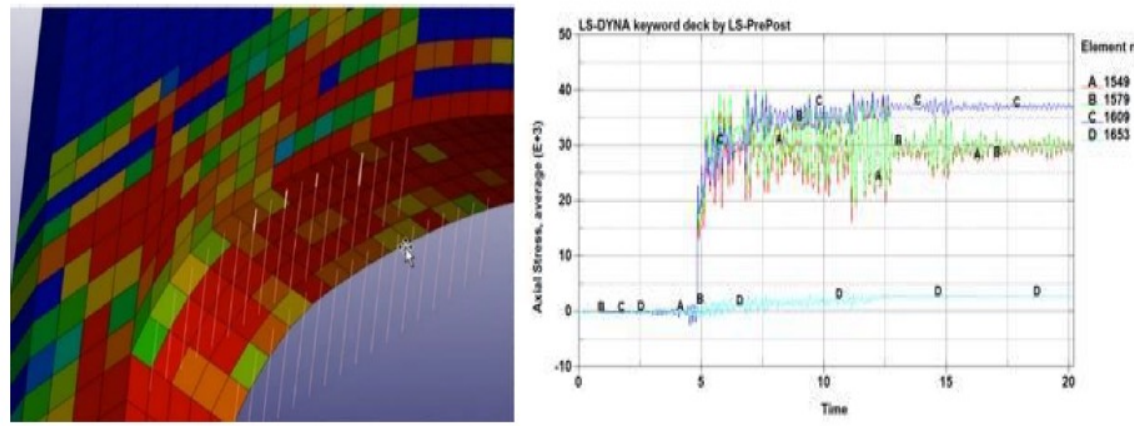
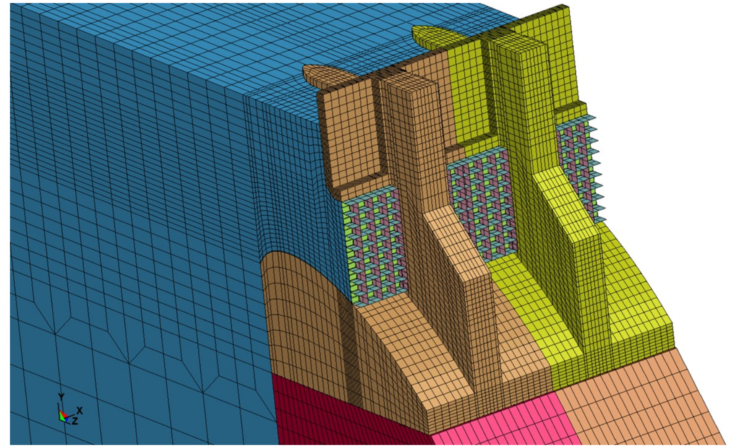


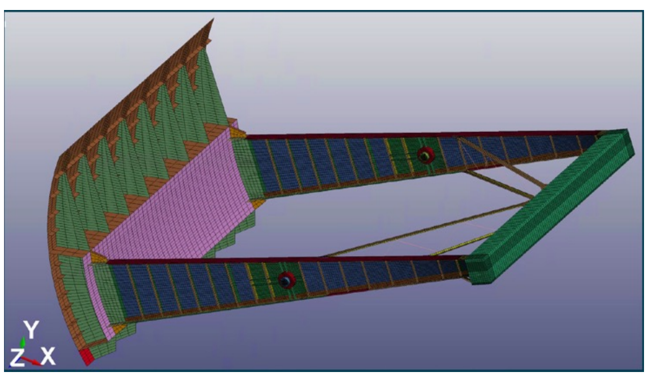
Figure 27.—Time history of representative reinforcement bars as highlighted (psi).

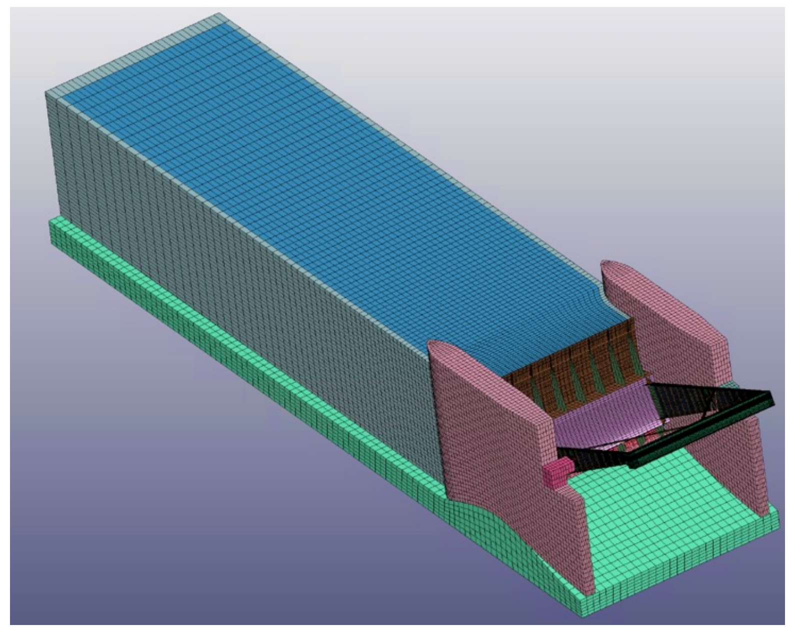
Gate (5)

When there are other factors beside hydro-dynamic load:

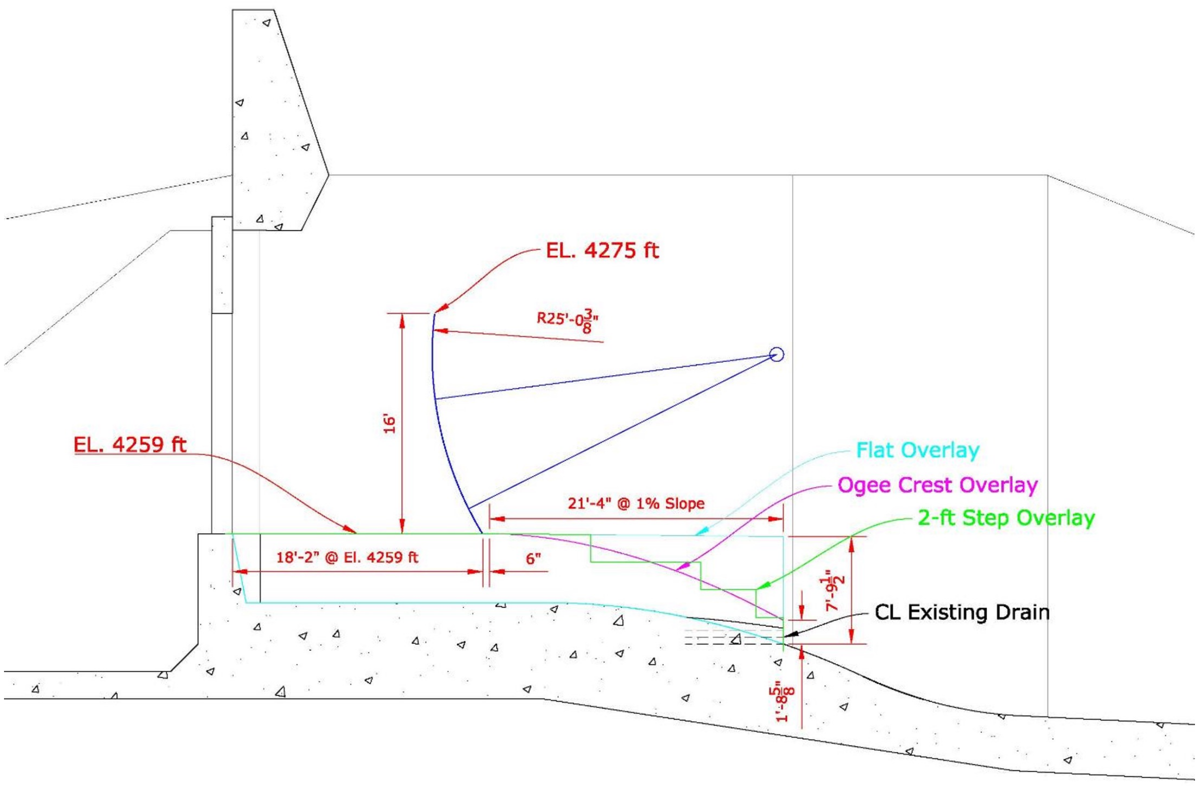
- Gate arm buckling
- Counterweight movement
- Nonlinear Steel Properties





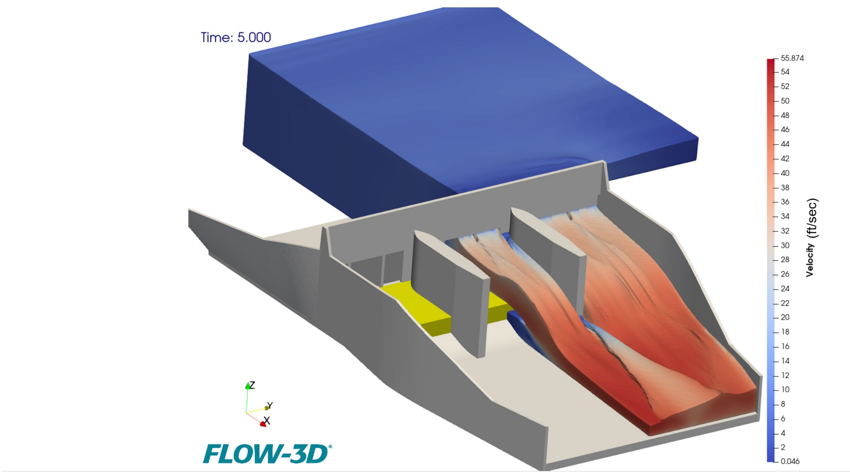


Spillway slab

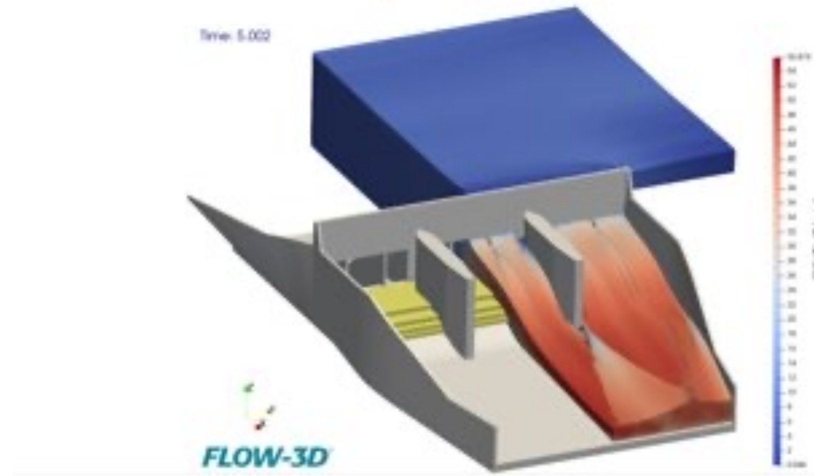


Flow Patterns Look Similar

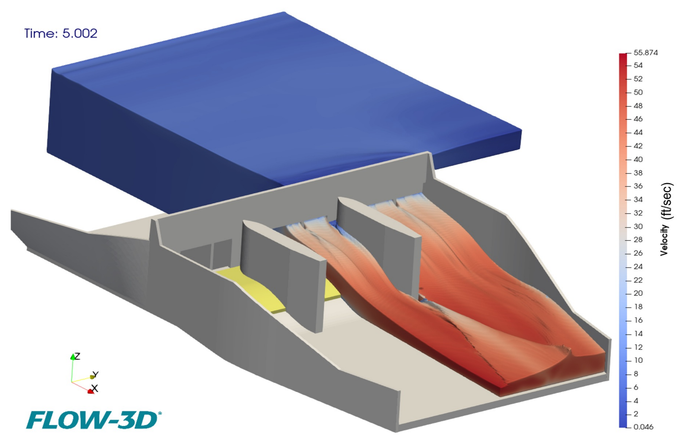
Results – Flat @ 56,000 cfs



Results – 2-ft Steps @ 56,000 cfs

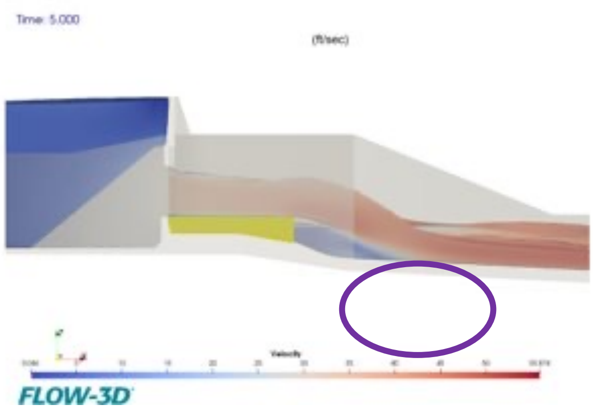


Results – Ogee @ 56,000 cfs

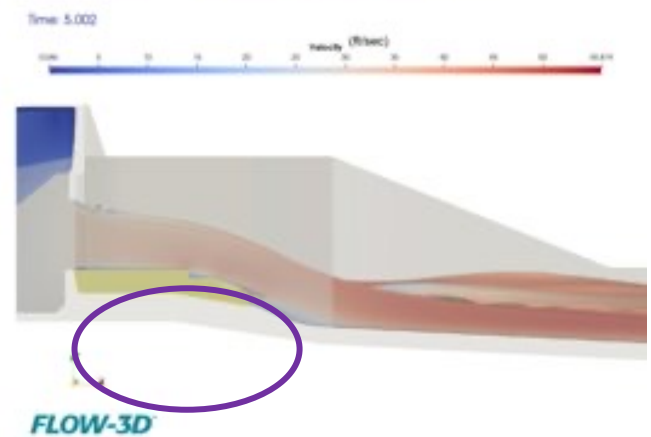


Spillway Slab

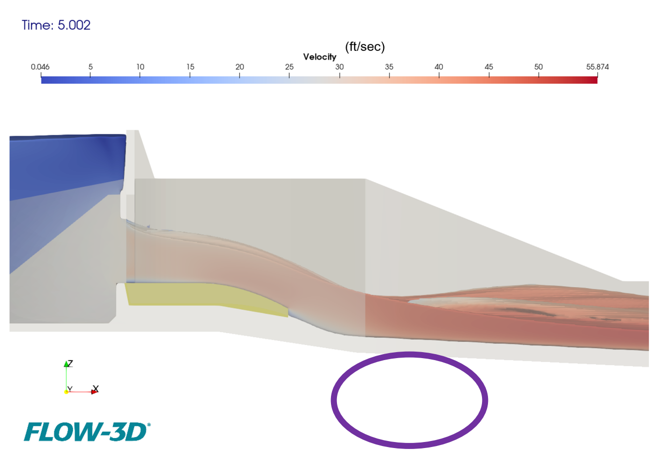
Results – Flat @ 56,000 cfs



Results – 2-ft Steps @ 56,000 cfs

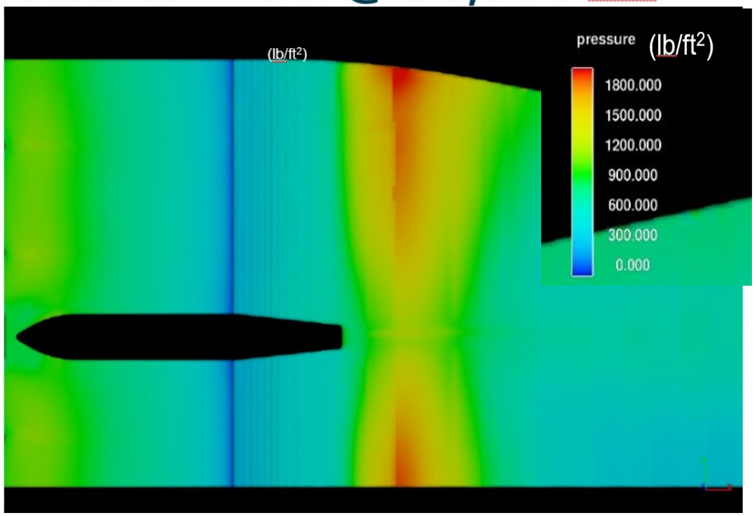


Results – Ogee @ 56,000 cfs

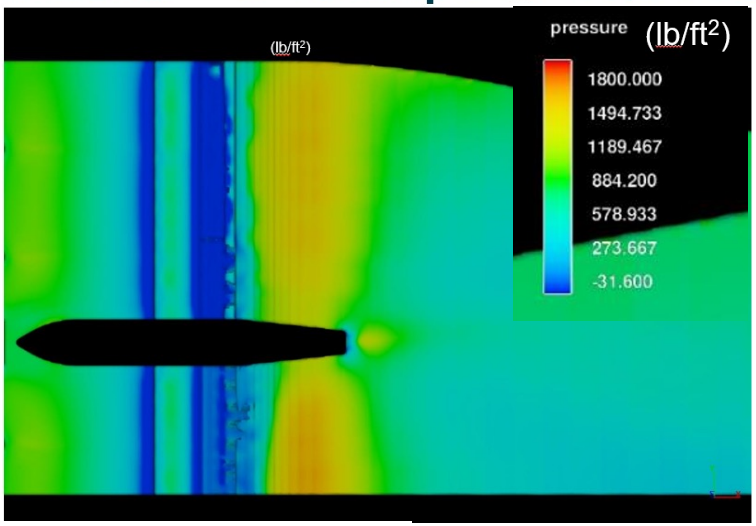


Spillway Slab (2)

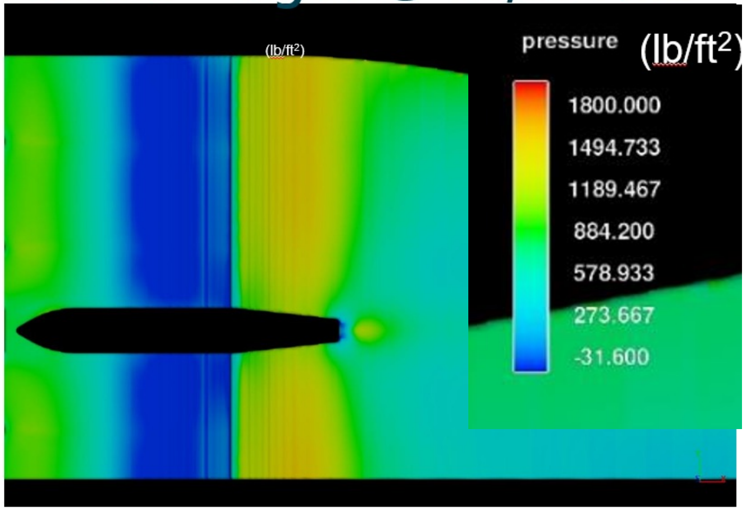
Results – Flat @ 56,000 cfs



Results – 2-ft Steps @ 56,000 cfs



Results – Ogee @ 56,000 cfs



FEMA

Other Potential Failure Modes

<u>Topic presented in DOI Safety training in 2018</u>

- Overtopping of the spillway chute walls
- Stagnation Pressure
- Foundation Erosion
- Hydraulic Jacking
- Concrete block sliding

(diversion dam/spillway block)