Best Practices in Dam and Levee Safety Risk Analysis

Seismic Spillway Pier (V-6) and Retaining Wall (V-7) Failure

22 April 2015

U.S. DEPARTMENT OF THE INTERIOR
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Seismic Failure of Walls and Piers

- This potential failure mode typically relates to gated spillway crest structures
- Similar potential failure modes in that both deal with the seismic response of reinforced concrete structures (Chapter V-1)
- Failure of walls or piers could lead to spillway gate failure
- Typically not an issue for uncontrolled spillways
- Without gates and water stored against the crest structure, uncontrolled release not likely to result from wall or pier failure
- Different in that walls are affected by soil backfill and piers are mostly loaded by inertial effects (cross canyon EQ)
- Similar in that both walls and piers are affected by hydrodynamic water loads on spillway gates (u/s-d/s EQ)

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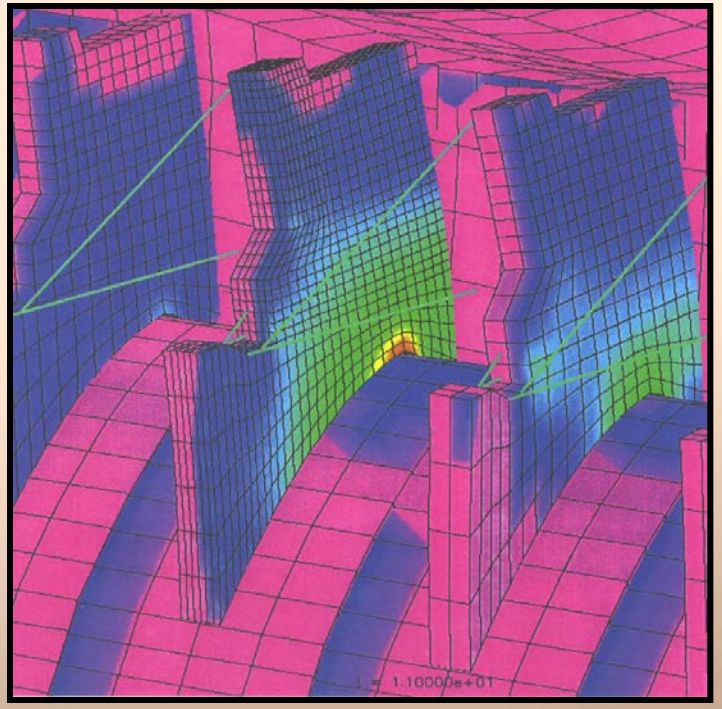
Seismic Spillway Pier PFM

- Applies to gated spillways with intermediate concrete piers
- Large inertial loads can be generated in spillway piers during seismic loading
- Spillway gates typically anchored within spillway piers, which transfers additional load to piers (static and hydrodynamic)
- Consider cross-canyon direction and upstream-downstream direction
- Pier failure or pier deflection can lead to gate failure

Seismic Spillway Pier PFM

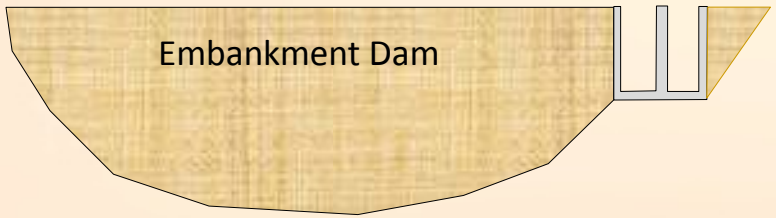


- No known case histories where pier failure resulted in uncontrolled release of the reservoir
- Analyses with large earthquake loadings have indicated potential for failure and Reclamation has modified piers at several dams

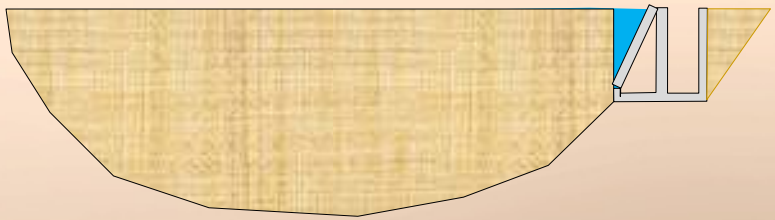
Seismic Spillway Wall PFM

- Wall failure can occur a number of different ways:
  - Wall collapses failing gate
    1. Wall loses structural integrity
    2. Wall collapses inwards
    3. Adjacent gate fails
    4. Uncontrolled release through spillway bay
  - Wall deflects excessively and damages gate
    1. Wall deflects excessively
    2. Unanticipated load on gate structural member(s)
    3. Gate buckles
    4. Uncontrolled release through spillway bay
  - Wall deflects creating seepage path
    1. Wall deflects sufficiently to create a gap between wall and adjacent embankment
    2. Seepage path established through gap
    3. Progressive erosion of adjacent embankment
    4. Breach of embankment

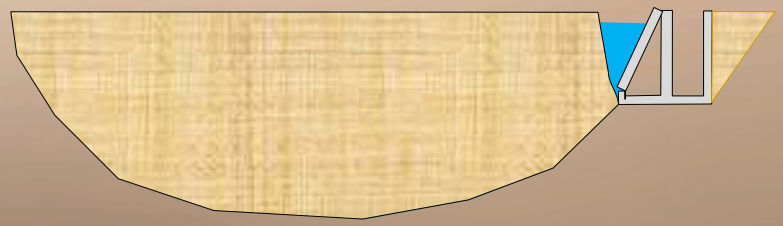
Seismic Spillway Wall PFM



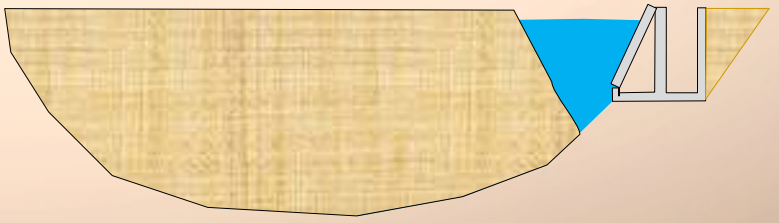
1 - Original Embankment and Gated Spillway



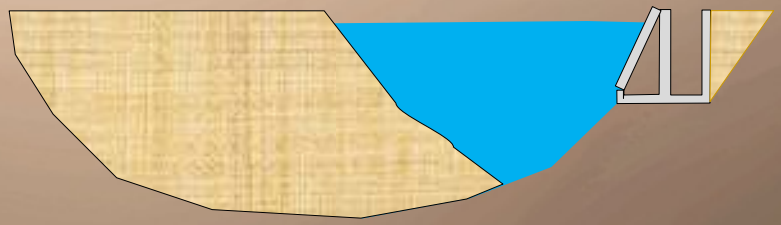
2 – Earthquake damages spillway wall producing upstream to downstream seepage path



3 – Embankment starts to scour



4 – Embankment continues to scour and cannot be stopped



5 – Embankment fails

Shi-Kang Dam

- Shi-Kang Dam is a buttress gravity dam located on the Tachia River
- Located about 30 miles north of the epicenter of the Chi-Chi earthquake (9/21/99)
- Chelungpu fault passed underneath spillway and ruptured during earthquake
- Differential movement through spillway was 29 feet
- PHA – 0.6g near dam
- But evidence that ground shaking at the site was not that intense

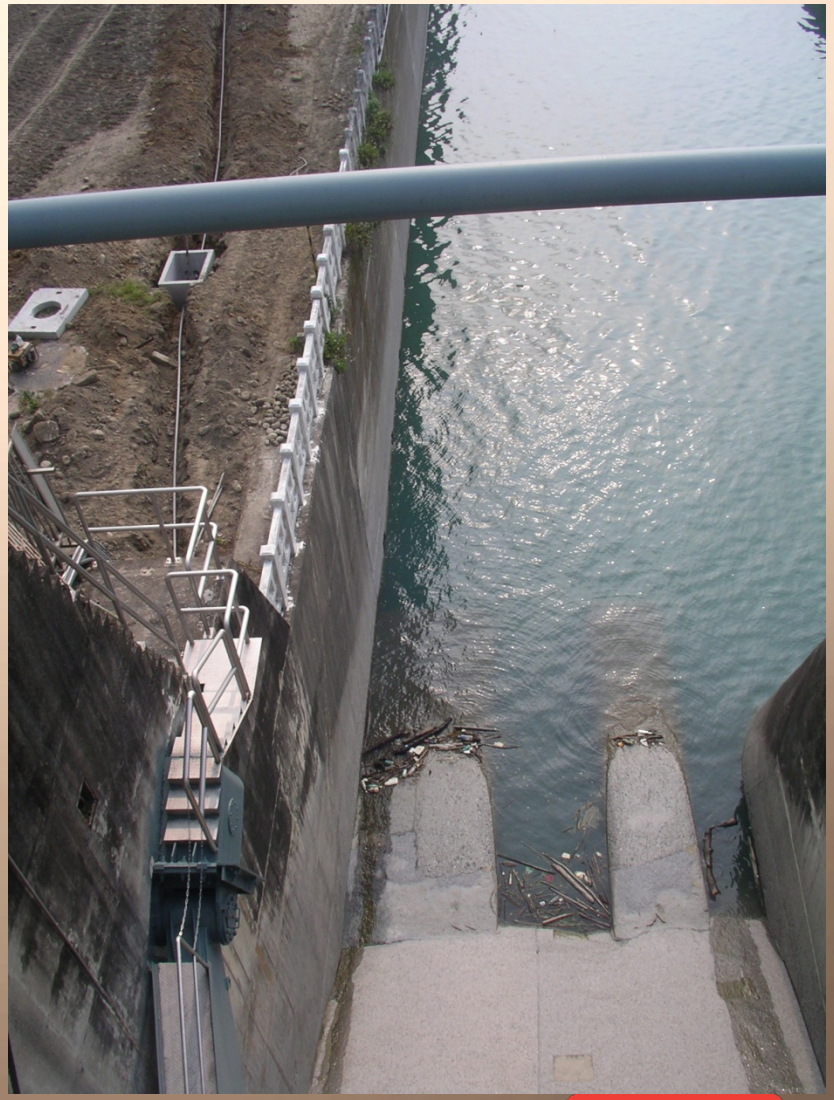
Shi-Kang Dam

Shi-Kang Dam

Shi-Kang Dam Spillway Wall

- Chute wall panel failed during 1999 earthquake
- Failure appears to be a shear failure through the counterforts
- No specific details are available for the structure
- Adequately designed and/or constructed?

Shi-Kang Dam



Austrian Dam Spillway

- Austrian Dam is a 200-foot high embankment dam constructed on Los Gatos Creek, near Los Gatos CA
- Concrete spillway located on right abutment of dam
- Austrian Dam was subjected to Loma Prieta earthquake on 10/17/89
- Estimated that PHA at site was up to 0.6g

Austrian Dam Spillway

- Austrian Dam settled and spread as a result of the earthquake – max settlement = 2.8 ft
- Downstream movement near spillway wall = 1.1 ft
- Embankment separated from spillway crest structure
- Spillway damage
  - cutoff walls were loaded and displaced
  - chute elongated about 1 foot as a result of embankment deformation
  - voids up to 6 inch voids created upstream of cutoff walls
  - chute walls deflected inward
  - potential seepage path created

Austrian Dam Spillway

Austrian Dam Spillway

Austrian Dam Spillway

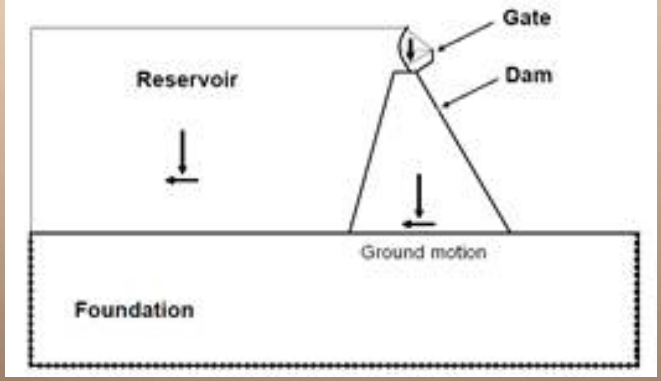
Key Factors – Walls and Piers

- Reservoir Water Surface Elevation
- Wall/Pier Geometry
- Moment Capacity
- Shear Capacity
- Seismic Hazard
- Spillway Bridges
- Gate Loads
- Trunnion Anchorage
- Number of Piers (piers only)
- Wall Backfill (walls only)
- Counterforted Walls (walls only)
- Seismic Earth Pressure (walls only)

Reinforced Concrete Failure Mechanisms – Chapter V-1

Reservoir Water Surface Elevation

- Reservoir water level on spillway crest structure is a key parameter for this potential failure mode
- Can affect loading on walls and piers
- Can affect access to seepage path behind wall
- Can affect consequences from wall or pier failure



Wall/Pier Geometry

- Wall/pier geometry affects seismic response
- Stiffer wall/pier may attract more load, while a flexible wall may relieve load through deflection
- Response depends on frequency of soil-wall system or pier and frequency content of earthquake
- Whether the crest structure is founded on rock or soil
- Configuration of an abutment slope above the spillway crest structure
- Orientation of the embankment with respect to the spillway crest structure

Moment and Shear Capacity

- Many Reclamation and USACE spillway structures have walls/piers that were not designed for current seismic loads
- Shear reinforcement is typically not provided in walls/piers
- Geometry and support conditions of the section
- Material properties of the reinforcement
- Material properties of the concrete
- Amount and detailing of reinforcement
- Type and duration of loading
- Loading in cross-canyon direction is typically the critical direction for walls; for piers loading in each direction (cross-canyon & u/s-d/s) may be important
- Location of the reinforced concrete members relative to the entire structure
- Reinforced concrete failure mechanisms section (Chapter V-1) provides specific guidance on estimating moment and shear capacity

Seismic Hazard

- Most spillway walls/piers have some reserve capacity beyond stress levels created by static loads
- Most walls/piers were not designed for significant seismic loading
- Some studies have indicated that retaining walls designed only for static loading may have enough reserve capacity to safely withstand seismic loadings between 0.4 and 0.5g
- Some Reclamation structures currently have PHA for 10,000 year earthquake level of > 1.0g
- Level of seismic loading in combination with static loading will determine level of overstress in wall/pier

Spillway Bridges

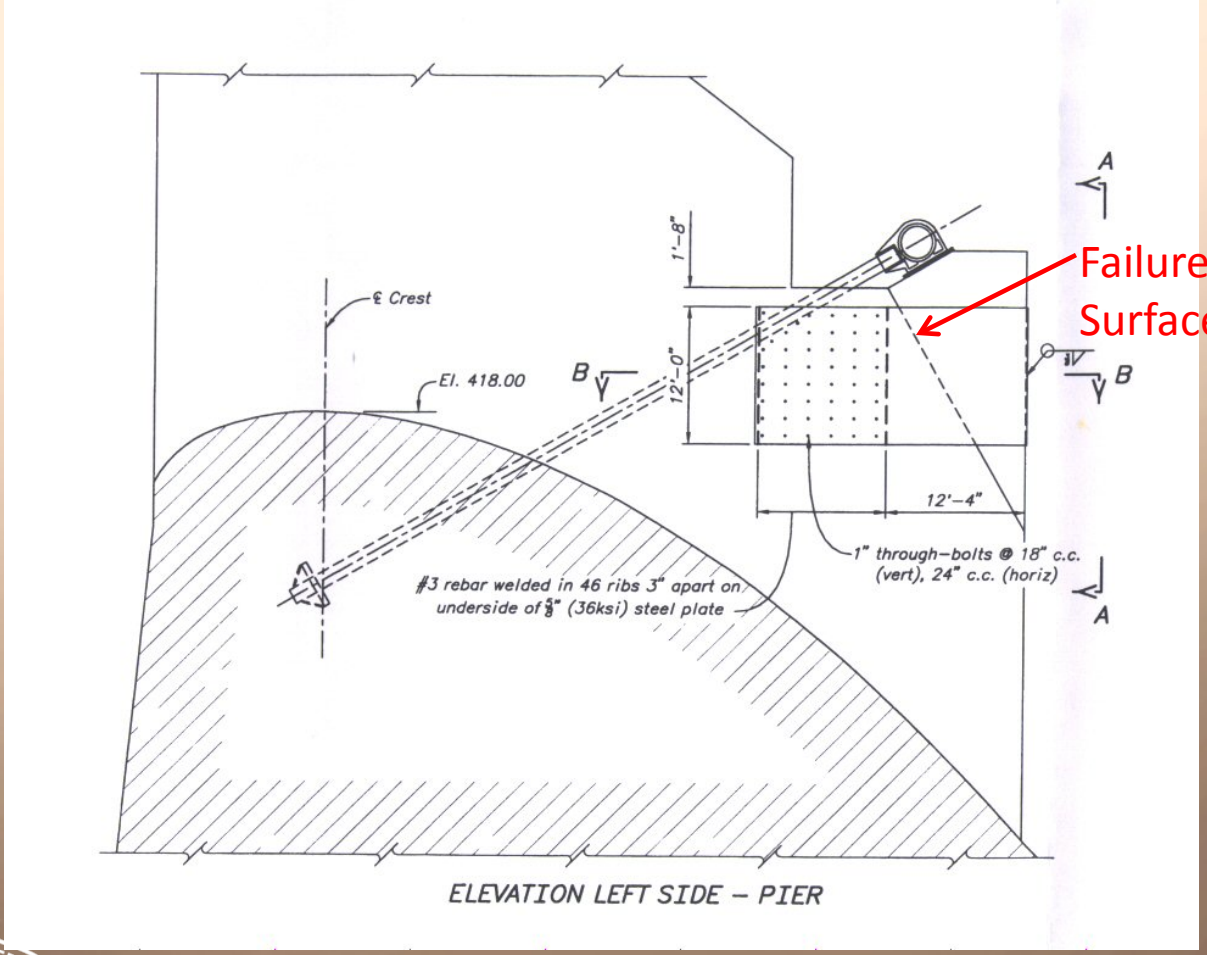
- Bridges are typically provided across the top of spillway crest structures – hoist decks and highway bridges
- Bridges may serve as struts for walls/piers but this needs to be verified
- Bridges can add inertial loads at top of piers
- Bridges can also fail during an earthquake and possibly impact gates

Spillway Hoist Deck

Gate Loads & Trunnion Anchorage

- Large hydrodynamic loads can be transferred from gates to walls/piers during an earthquake
- Anchorage is evaluated for static and hydrodynamic loads on gate – assuming full load is transferred to trunnion and trunnion anchorage
- Pseudo-static analysis may indicate that trunnion anchorage is stressed to levels beyond ultimate capacity
- A time-history analysis may indicate that anchorage can not strain enough to fail (for anchors with unbonded free length)
- Loads transmitted from gates into piers/walls can lead to sliding or local overstressing concrete

Spillway Pier – Trunnion Shear Failure



Unique Considerations for Piers

Number of Piers

- Multiple piers increase the probability of pier failure
- Failure of one pier will most likely lead to failure of two gates
- Multiple pier failure will increase the breach outflow and downstream consequences
- If multiple pier failures occur, consequences will be a function of failure configuration (series vs. staggered)

Multiple Piers - Pascal's Triangle

Number of Gates																											
0															1												
1														1		1											
2													1		2		1										
3												1		3		3		1									
4											1		4		6		4		1								
5										1		5		10		10		5		1							
6									1		6		15		20		15		6		1						
7								1		7		21		35		35		21		7		1					
8							1		8		28		56		70		56		28		8		1				
9						1		9		36		84		126		126		84		36		9		1			
10					1		10		45		120		210		252		210		120		45		10		1		
11				1		11		55		165		330		462		462		330		165		55		11		1	
12			1		12		66		220		495		792		924		792		495		220		66		12		1
13		1		13		78		286		715		1287		1716		1716		1287		715		286		78		13		1
14	1		14		91		364		1001		2002		3003		3432		3003		2002		1001		364		91		14

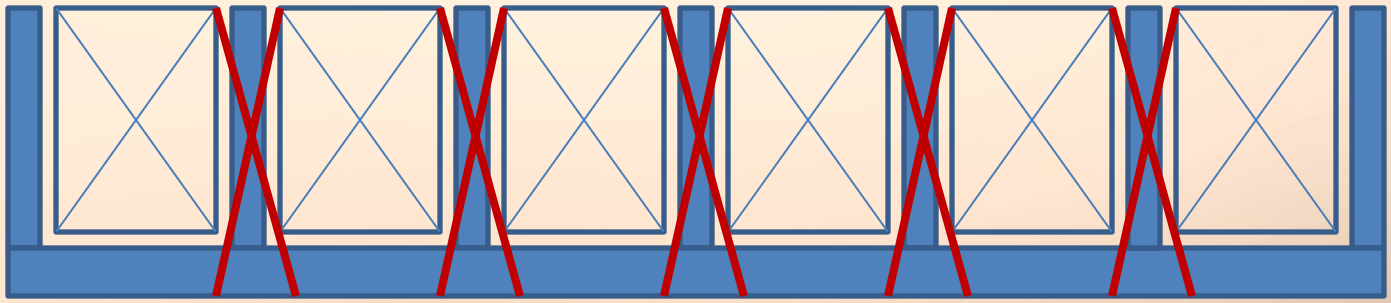
Figure 4 - Pascals's Triangle for Muptiple Gate Failure Probability Coefficients

Multiple Pier Estimates

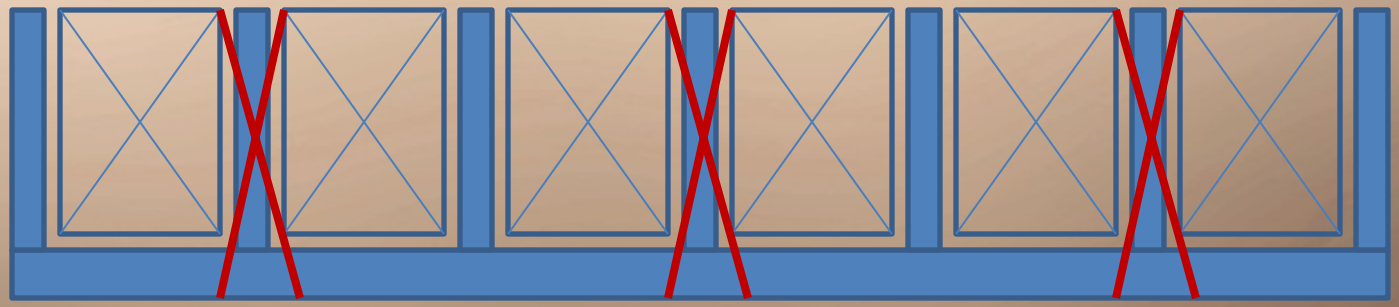
Probability for Single Pier Failure →		0.001	0.05	0.16	0.94
No. of Piers Failing	Equation for "x" Piers Failing	Probability for "x" Piers Failing	Probability for "x" Piers Failing	Probability for "x" Piers Failing	Probability for "x" Piers Failing
0	$1P^0(1-P)^5$	0.995	0.774	0.418	7.8E-7
1	$5P^1(1-P)^4$	0.005	0.204	0.398	6.0E-05
2	$10P^2(1-P)^3$	1.0E-05	0.021	0.152	1.9E-03
3	$10P^3(1-P)^2$	1.0E-08	0.001	0.029	0.03
4	$5P^4(1-P)^1$	5.0E-12	3.0E-05	0.003	0.234
5	$1P^5(1-P)^0$	1.0E-15	3.0E-07	1.0E-04	0.734
Total Probability of One or More Piers Failing		0.005	0.226	0.582	1.00

Multiple Pier Estimates

n+1 Failure Scenario



2n Failure Scenario



Pier Failure – n+1 (P=0.16)

Number of Piers Failing	Probability of Failure Equations	Probability (P_x) of (x) Piers Failing	Expected Life Loss Value	Life Loss for (x) Piers Failing x (P_x)
1	$P_1 = 5(P)^1(1-P)^4$	0.398	16*	6.37
2	$P_2 = 10(P)^2(1-P)^3$	0.152	23*	3.50
3	$P_3 = 10(P)^3(1-P)^2$	0.029	30*	0.87
4	$P_4 = 5(P)^4(1-P)^1$	0.003	147	0.44
5	$P_5 = 1(P)^5(1-P)^0$	1.0E-04	164	0.02
Totals		**0.58**		**11**

Weighted Ave Loss of Life = 11/0.58 = 19 people

Pier Failure – 2n (P=0.16)

Number of Piers Failing	Probability of Failure Equations	Probability (P_x) of (x) Piers Failing	Expected Life Loss Value	Life Loss for (x) Piers Failing x (P_x)
1	$P_1 = 5(P)^1(1-P)^4$	0.398	16*	6.37
2	$P_2 = 10(P)^2(1-P)^3$	0.152	30*	4.56
3	$P_3 = 10(P)^3(1-P)^2$	0.029	164	4.76
4	$P_4 = 5(P)^4(1-P)^1$	0.003	164	0.49
5	$P_5 = 1(P)^5(1-P)^0$	1.0E-04	164	0.02
Totals		**0.58**		**16**

Weighted Ave Loss of Life = 16/0.58 = 28 people

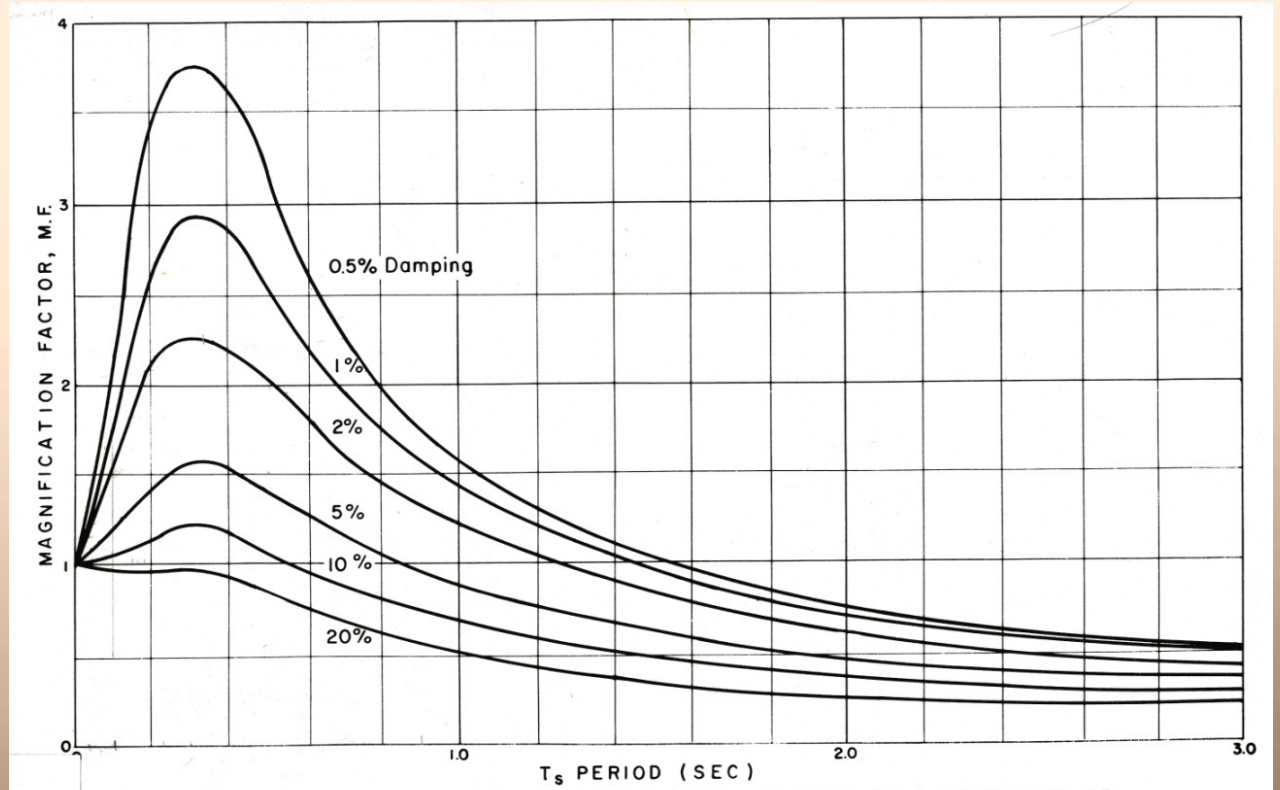
Additional Considerations for Seismic Spillway Pier Failure

- If reservoir is only up on the gates for limited durations may be able to make the case that failure probability is remote
- Simple pseudo-static analysis can be used to evaluate moment and shears
- A time history analysis will provide a more complete picture of:
  - the extent of overstressing
  - the number of overstress excursions
- Can model non-linear behavior with finite element modeling

Simplified Pier Analysis

- Pseudo-static approach that estimates the inertial loads during an earthquake
- Loading at base = **150 x B x α_h**
- Loading at top = **150 x B x α_h x MF**
  - 150 = unit weight of concrete (lb/ft^3)
  - B = base thickness of wall (ft)
  - α_h = horizontal ground acceleration (g)
  - MF = magnification factor
- **T_s = FH^2/B** = estimated fundamental period
- Use 2 percent damping (conservative, 5 percent is typical but response spectra are based on limited sites in California and Washington)

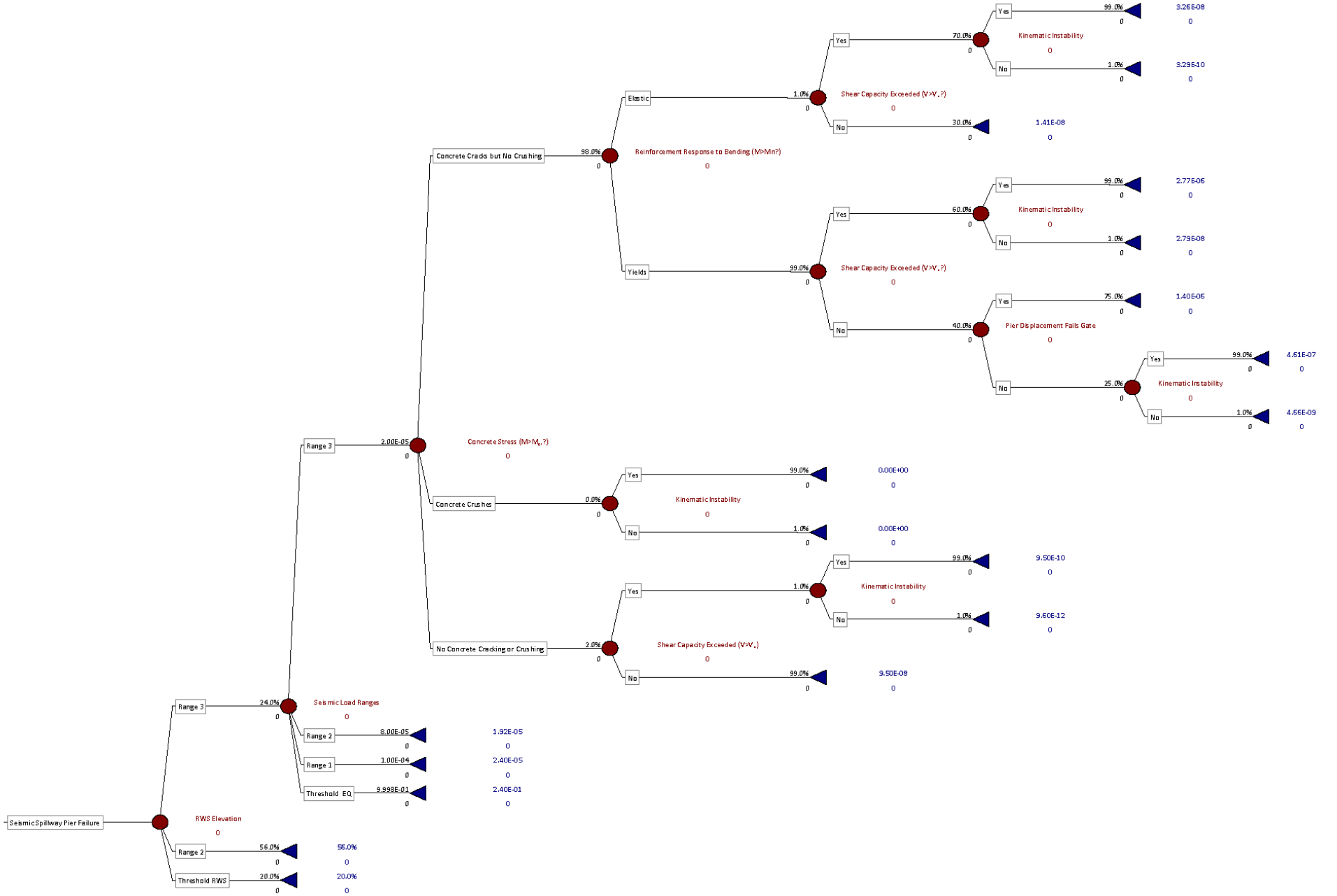
Magnification Factor for Seismic Piers



DESIGN SPECTRUM GIVING MAGNIFICATION FACTOR AS A FUNCTION OF PERIOD AND DAMPING; ARBITRARY SCALE

AFTER HOUSNER-CHAPTER 5 OF "EARTHQUAKE ENGINEERING," BY WIEGEL

Example Event Tree



Unique Considerations for Walls

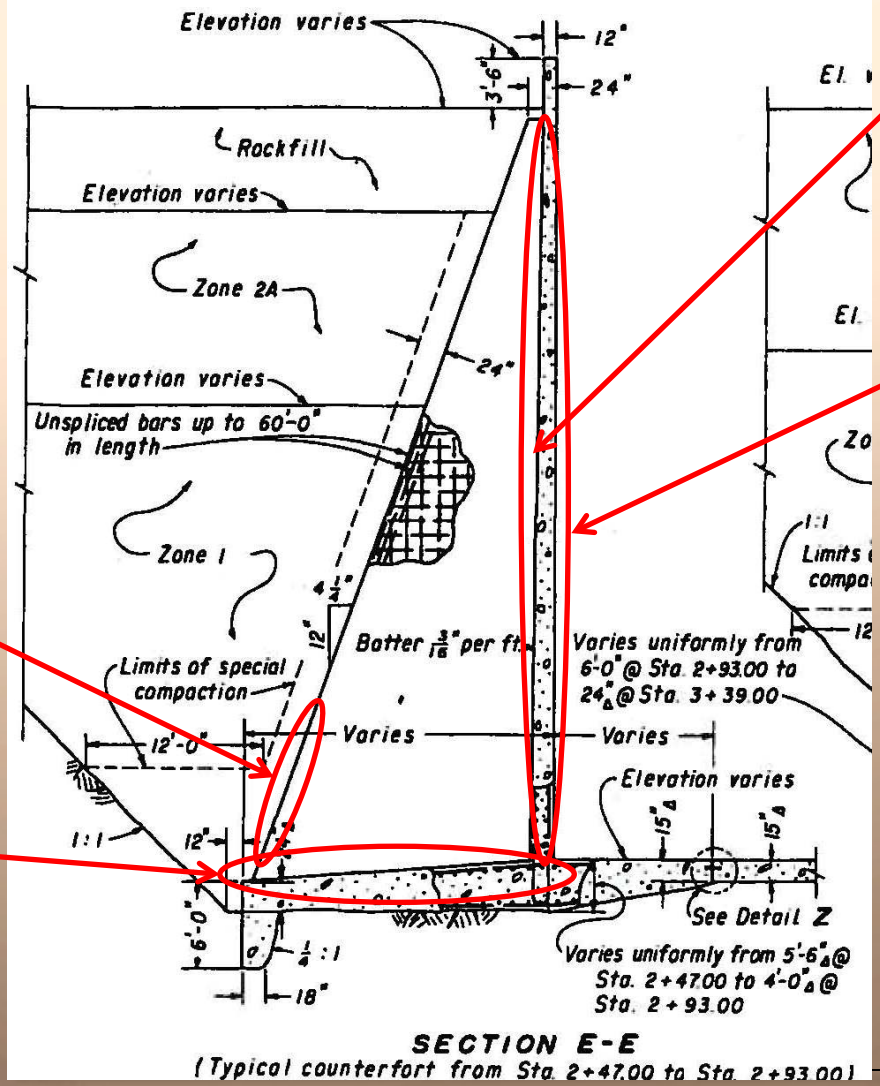
Wall Backfill

- Properties of wall backfill important component in determining seismic earth pressures acting on crest structure walls
- Saturation level influences static and seismic earth pressures on the wall
- Saturation level can be an indicator of ability of water to move through soil and develop a seepage path for potential initiation of internal erosion

Counterforted Walls

- Many spillway crest structure walls are counterforted
- Counterforted walls are more complicated to analyze than cantilevered walls
- Counterforted walls can fail through a number of mechanisms, involving the counterforts, the wall panels and the connection between the two

Counterforted Wall – Failure Mechanisms



counterfort moment failure (u/s-d/s)

wall panel moment or shear failure (cross-canyon)

counterfort moment failure (cross-canyon)

counterfort shear failure (cross-canyon)

Seismic Earth Pressure

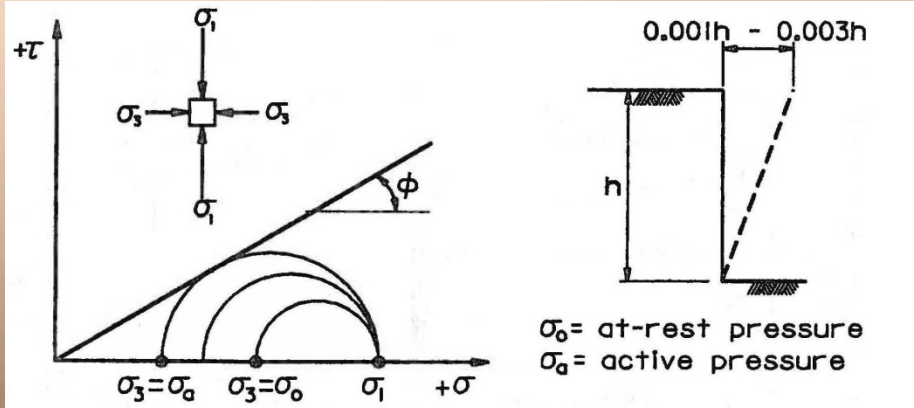
- Seismic earth pressure is the critical loading mechanism for spillway walls – combination of static and dynamic earth pressures
- Related to interaction of spillway wall and backfill
- Affected by spillway crest structure foundation (rock or soil)

Earth Pressure

- Seismic soil loadings are related to earth pressure theory and the state of the wall backfill prior to and during the earthquake

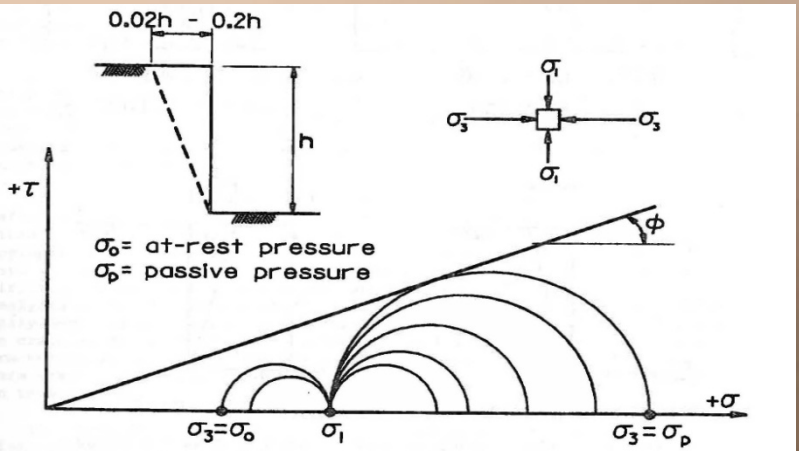
Earth Pressure

- Rankine Theory of Earth Pressure
  - Soil mass transformed from at-rest state to state of plastic equilibrium
  - Plastic equilibrium is condition where every point in a soil mass is on the verge of failure
  - Shear stress at failure defined by Mohr-Coulomb theory



a. Development of active earth pressure

$K_A = \tan^2(45-\Phi/2)$



b. Development of passive earth pressure

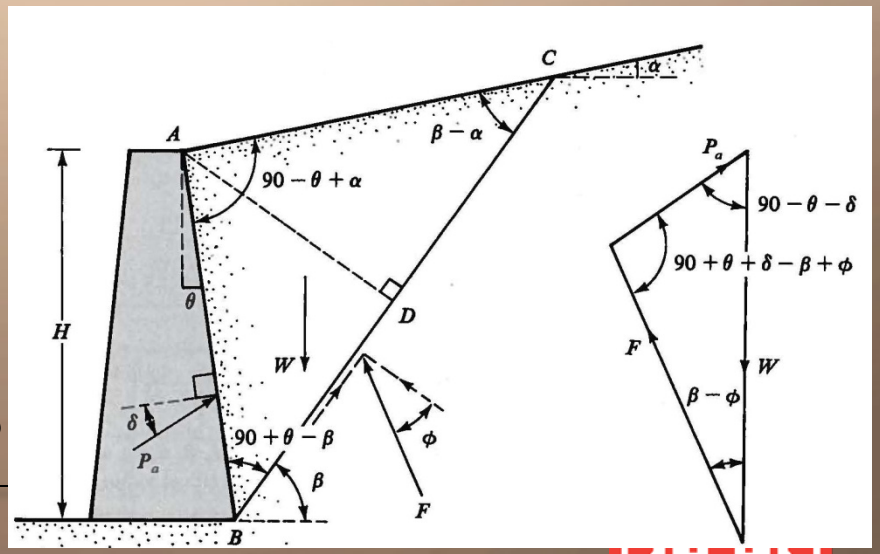
$K_P = \tan^2(45+\Phi/2)$

Earth Pressure

- Coulomb Wedge Theory
  - Considers equilibrium of forces acting on a soil wedge without considering the state of stress within the backfill
  - Wedge theory assumes a linear failure surface within the backfill and full mobilization of the shear strength along the failure surface
  - Analysis may consider:
    - ✓ Interface friction between the wall and the backfill (δ)
    - ✓ Slope of the backfill (α)
    - ✓ Batter of the wall (θ)

$$K_a = \frac{\cos^2(\phi - \theta)}{\cos^2\theta \cdot \cos(\delta + \theta)\left[1 + \sqrt{\frac{\sin(\delta + \phi) \cdot \sin(\phi - \alpha)}{\cos(\delta + \theta) \cdot \cos(\theta - \alpha)}}\right]^2}$$

$$K_p = \frac{\cos^2(\phi + \theta)}{\cos^2\theta \cos(\delta - \theta)\left[1 - \sqrt{\frac{\sin(\phi - \delta) \sin(\phi + \alpha)}{\cos(\delta - \theta) \cos(\alpha - \theta)}}\right]^2}$$



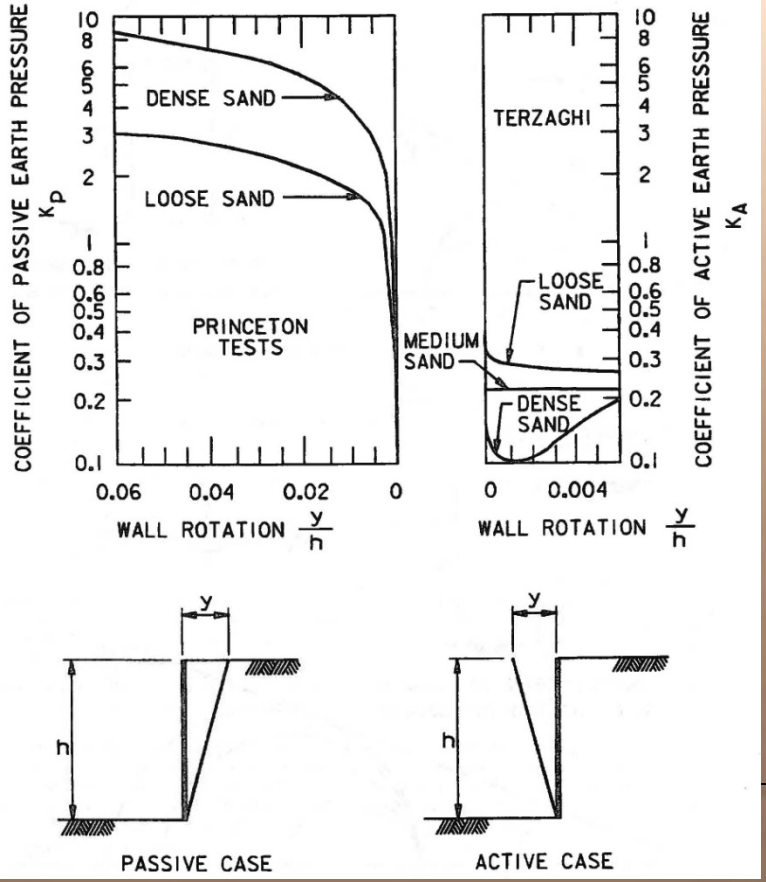
What is K_A & K_P when δ, α, and θ = 0° ?

Earth Pressure

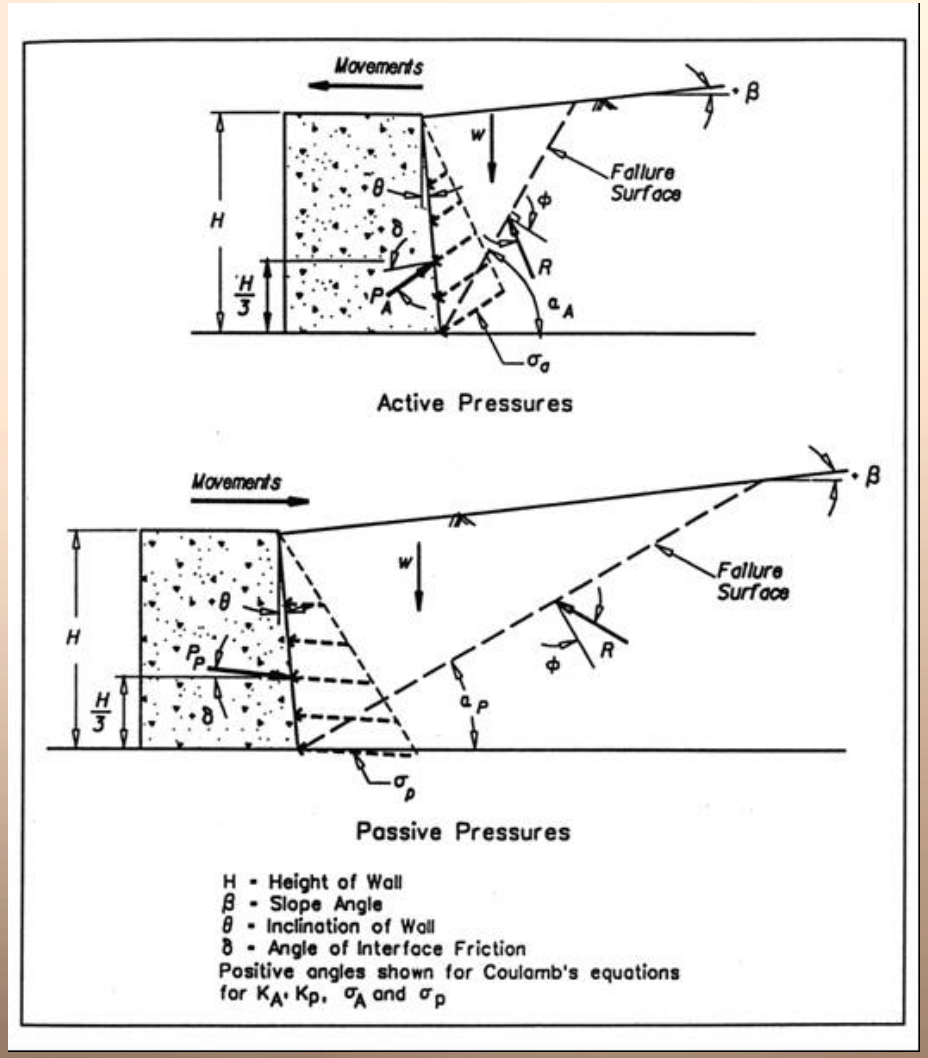
- Others have developed relationships for active and passive earth pressure, assuming logarithmic failure surface
- Rankine's Theory, Coulomb's Wedge Theory and logarithmic spiral procedure result in similar values for active and passive earth pressures when no friction assumed between wall and backfill
- For interface angles greater than 0, Coulomb's Wedge Theory and logarithmic spiral procedure result in nearly the same value for active earth pressures

Earth Pressure

- Active Earth Pressure K_A – wall moves away from soil
- Passive Earth Pressure K_P – wall moves into soil
- At-rest Earth Pressure K_o – wall is rigid and backfill does not yield
- $K_A < K_o < K_P$



Active/Passive Condition



- Horizontal effective stress, $\sigma = K\gamma H$
- Resultant static earth force, $P = \frac{1}{2} K\gamma H^2$

Seismic Earth Pressure

- Mononobe-Okabe

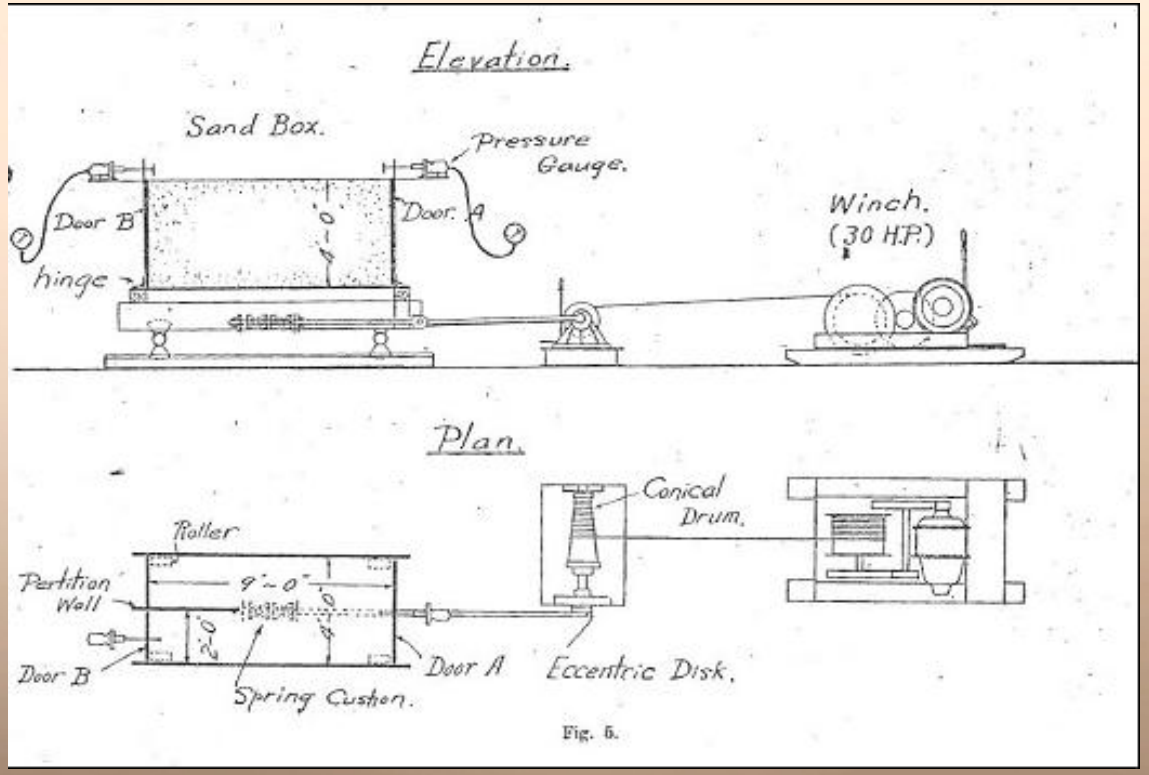
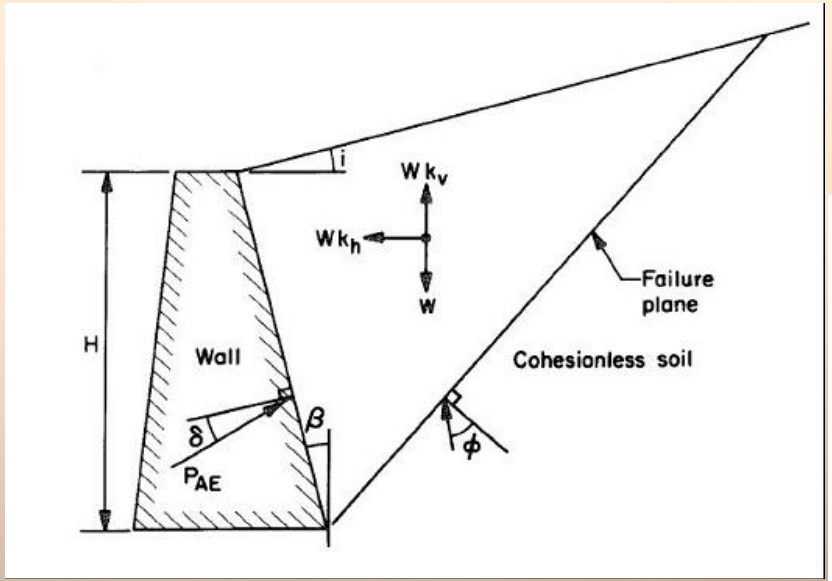


Fig. 5.

- Mononobe-Matsuo 1929 test in Japan
- Rigid, small scale, 1g shake table
- Box (9'Lx4'Wx4'H) filled with loose dry sands on rollers
- Winch driven by 30 HP electric motor
- Horizontal simple harmonic motion
- Hydraulic pressure gauges mounted on top to measure earth pressures

Mononobe-Okabe Relationship



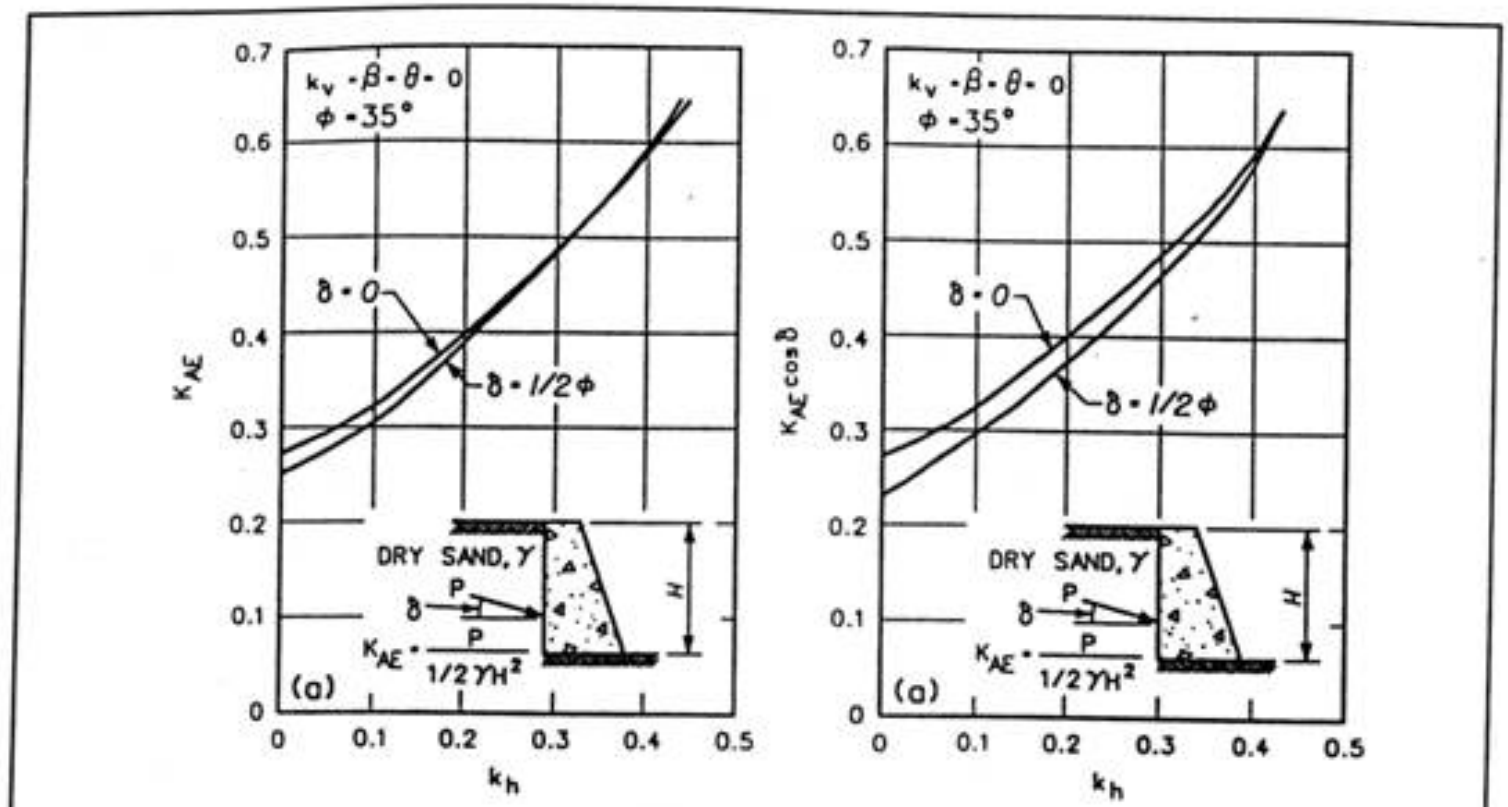
$$P_{AE} := \frac{1}{2} \cdot \gamma \cdot H^2 \cdot (1 - k_v) \cdot K_{AE}$$

$$K_{AE} := \frac{\cos(\phi - \psi - \beta)^2}{\cos(\psi) \cdot \cos(\beta)^2 \cdot \cos(\delta + \beta + \psi) \left(1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - i - \psi)}{\cos(\delta + \beta + \psi) \cdot \cos(i - \beta)}}\right)^2}$$

P_{AE} = maximum dynamic active force per unit width of the wall
K_{AE} = total lateral earth pressure coefficient
γ = unit weight of the soil
H = height of the wall
φ = angle of internal friction of the soil
δ = angle of wall friction
i = slope of ground surface behind the wall
β = slope of the wall relative to the vertical
k_h = horizontal wedge acceleration divided by g
k_v = vertical wedge acceleration divided by g

$$\psi := \operatorname{atan}\left(\frac{k_h}{1 - k_v}\right)$$

Values of K_{AE}



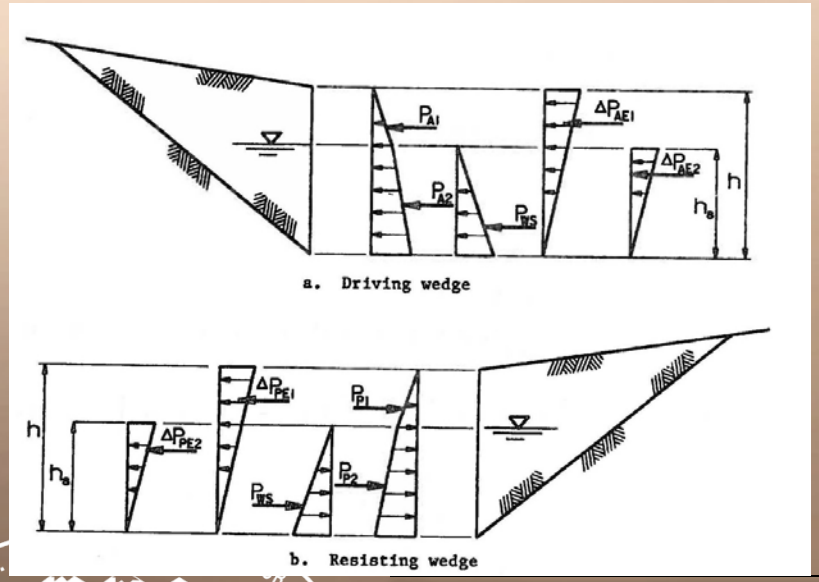
From Seed and Whitman (1970).

Mononobe-Okabe Assumptions

- Yielding wall with active pressures
- Cohesionless backfill
- Soil satisfies Mohr-Coulomb failure criterion
- Failure plane in backfill occurs along inclined angle and passes through the toe of the wall
- No liquefaction
- Soil wedge behaves as a rigid body and accelerations are constant throughout the mass
- Backfill completely above or completely below the water table
- Limitation - $\phi \geq \psi$

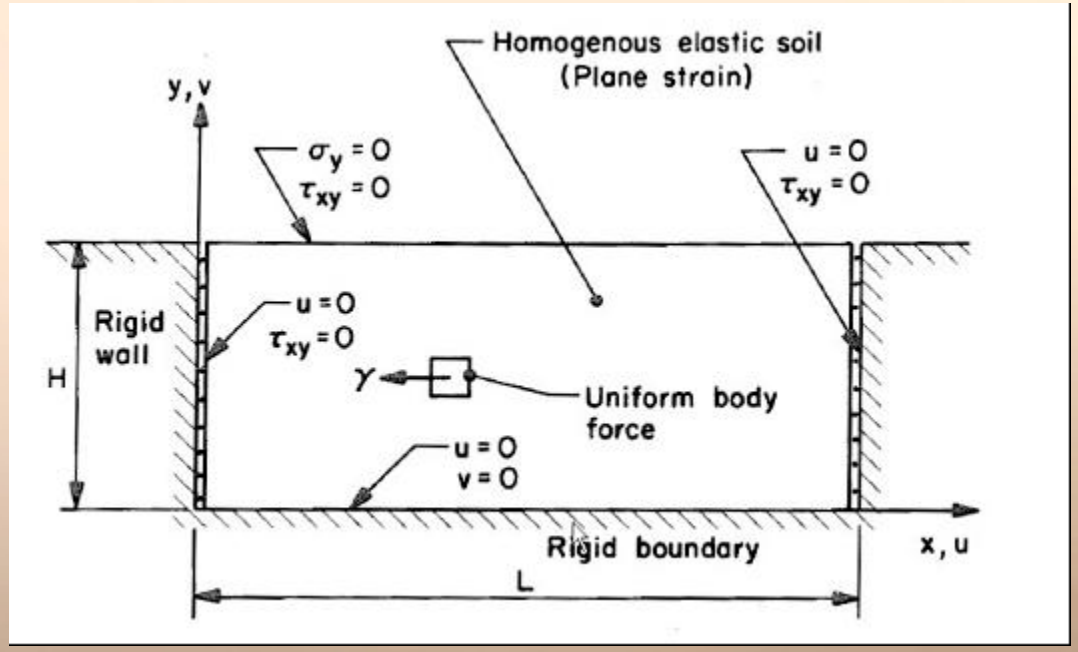
Mononobe-Okabe Studies

- Seed and Whitman - 1970 parametric studies on δ, ϕ, β
  - Consistent M-O total dynamic force
  - $P_{AE} = P_A + \Delta P_{AE} \quad \therefore \quad K_{AE} = K_A + \Delta K_{AE}$
  - Inverted triangle distribution – ΔP_{AE} @ 2/3H
  - Use 85% of PGA for design
  - Adequate static design may provide sufficient seismic capacity in many cases



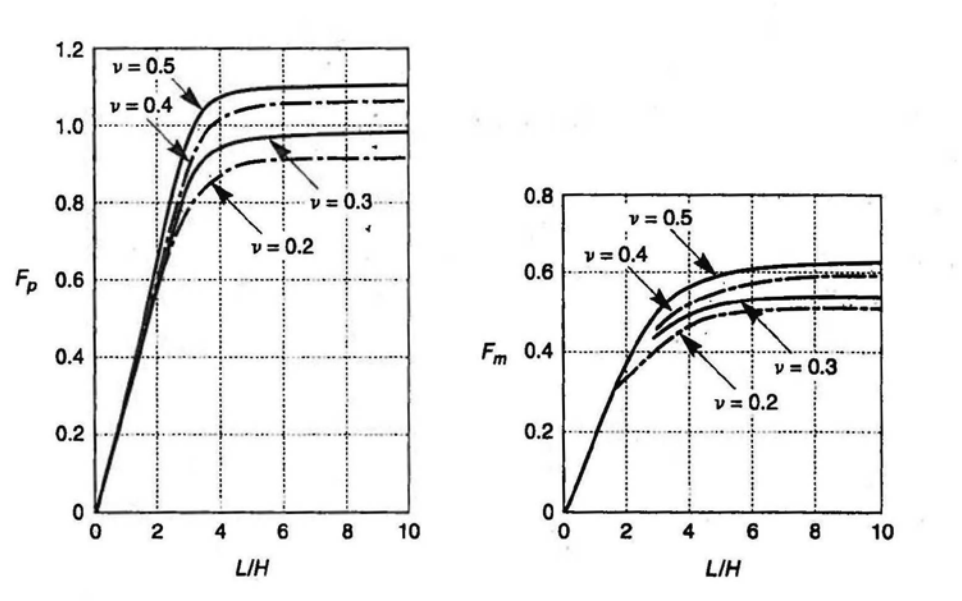
- Ebeling and Morrison – 1992 study of effects of saturated backfill
  - Saturation needs consideration
  - Function of whether pore water pressures increase within the backfill during an earthquake
  - Function of whether the water in the backfill is restrained or free, which is a function of the permeability of the backfill

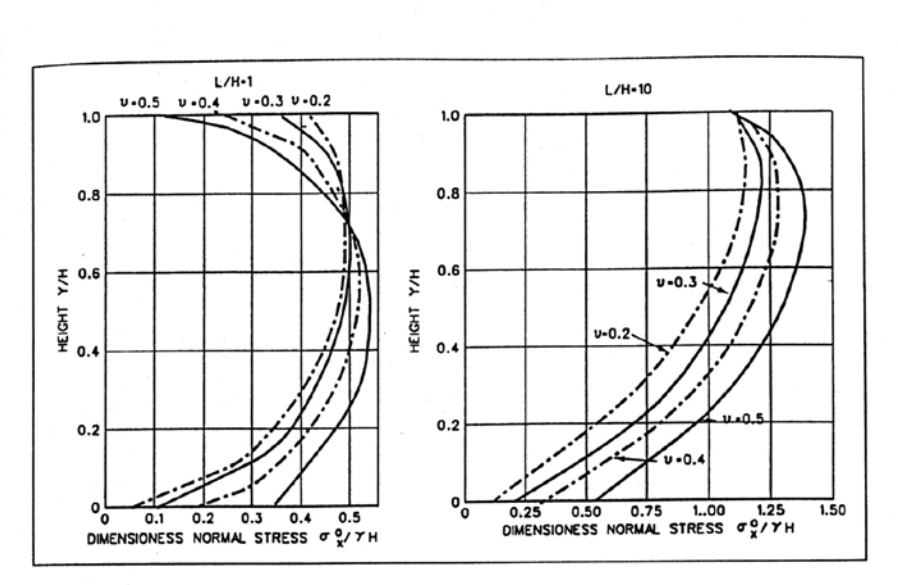
Wood's Solution



- Elastic method developed in 1973
- Rigid, non-yielding walls such as basement walls
- Displacements generate soil stresses in the elastic range
- Elastic wave solutions
- Upper bound ≅ 2 to 3 x M-O
- Dynamic earth pressures must be added to static earth pressures
- Function of soil Poisson's ratio
- Function of L/H
- Not limited for large response accelerations
- Shaking frequency << fundamental backfill frequency
- Normalized solutions

Wood's Solution





$\Delta P_{AE} = \gamma H^2 k_h F_p$ – dynamic thrust

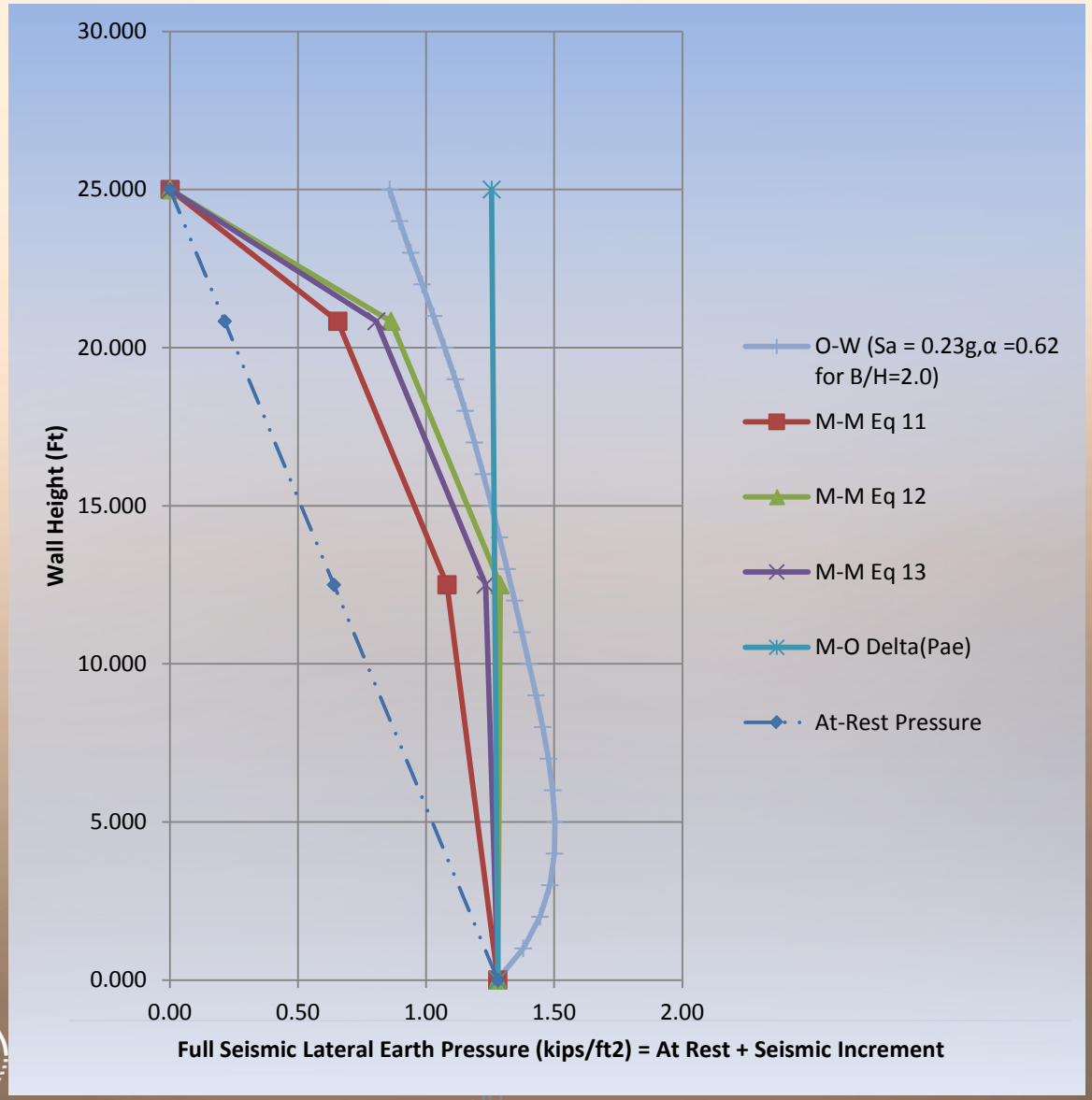
$\Delta M_{AE} = \gamma H^3 k_h F_m$ – dynamic overturning moment

Parabolic Distribution - ΔP_{AE} @ 0.55 – 0.65H

Other Dynamic SSI Methods

- Ostadan and White Method (1998)
  - Frequency content of the earthquake is fully considered
- Maleki and Mahjoubi (2010)
  - Focused on the natural period of the soil-wall system

Other Dynamic SSI Methods



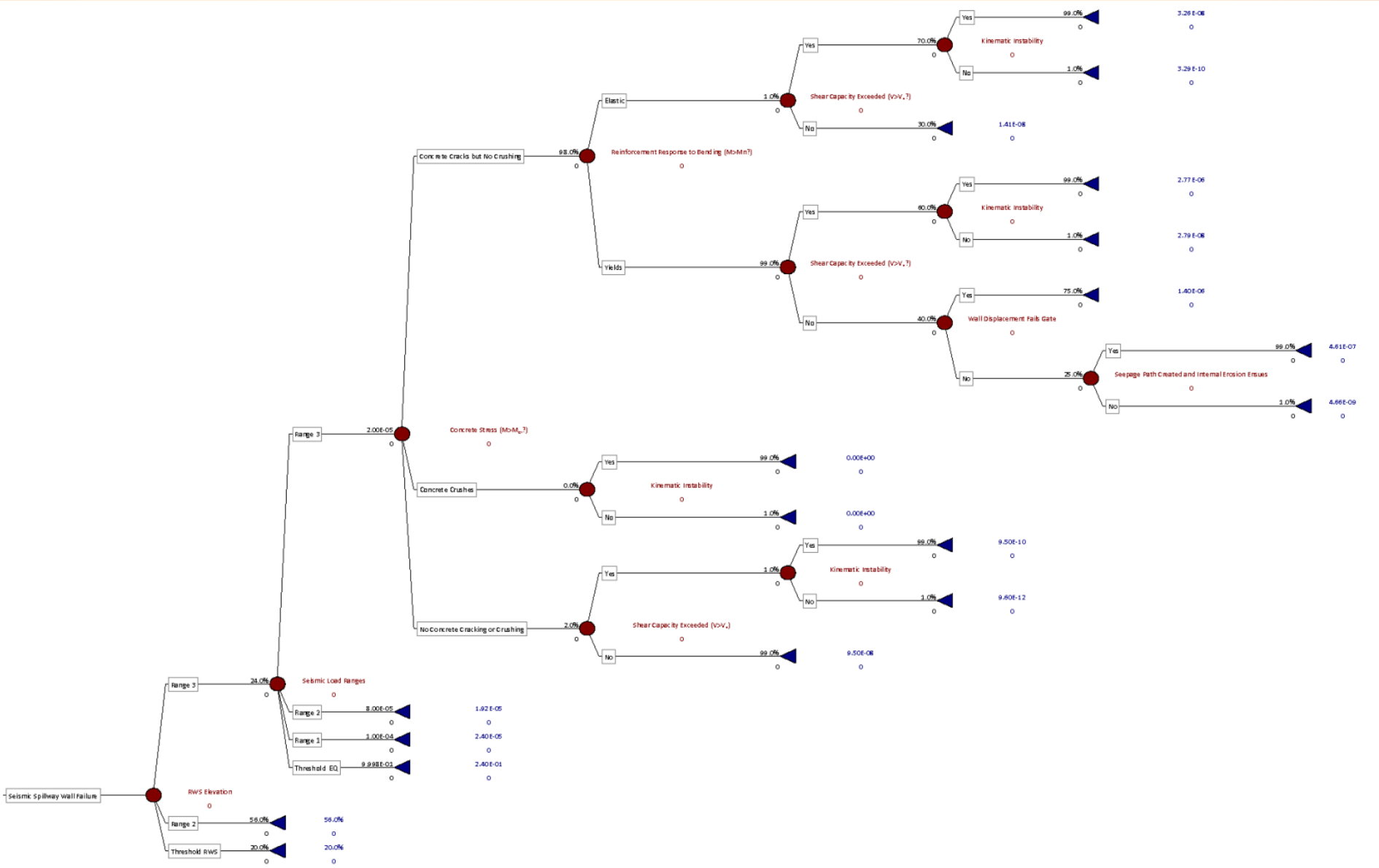
Spillway Wall Failure Progression

Seismic Failure of Spillway Crest Structure Walls → Seismic Load → Wall Deflects or Fails → Seepage Path Created → Continuous U/S-D/S Crack → Erosion Progresses and Breach Forms

Failed Spillway Wall at Shi-Kang Dam

Gated Spillway Adjacent to an Embankment Dam

Example Event Tree



Finite Element Analyses

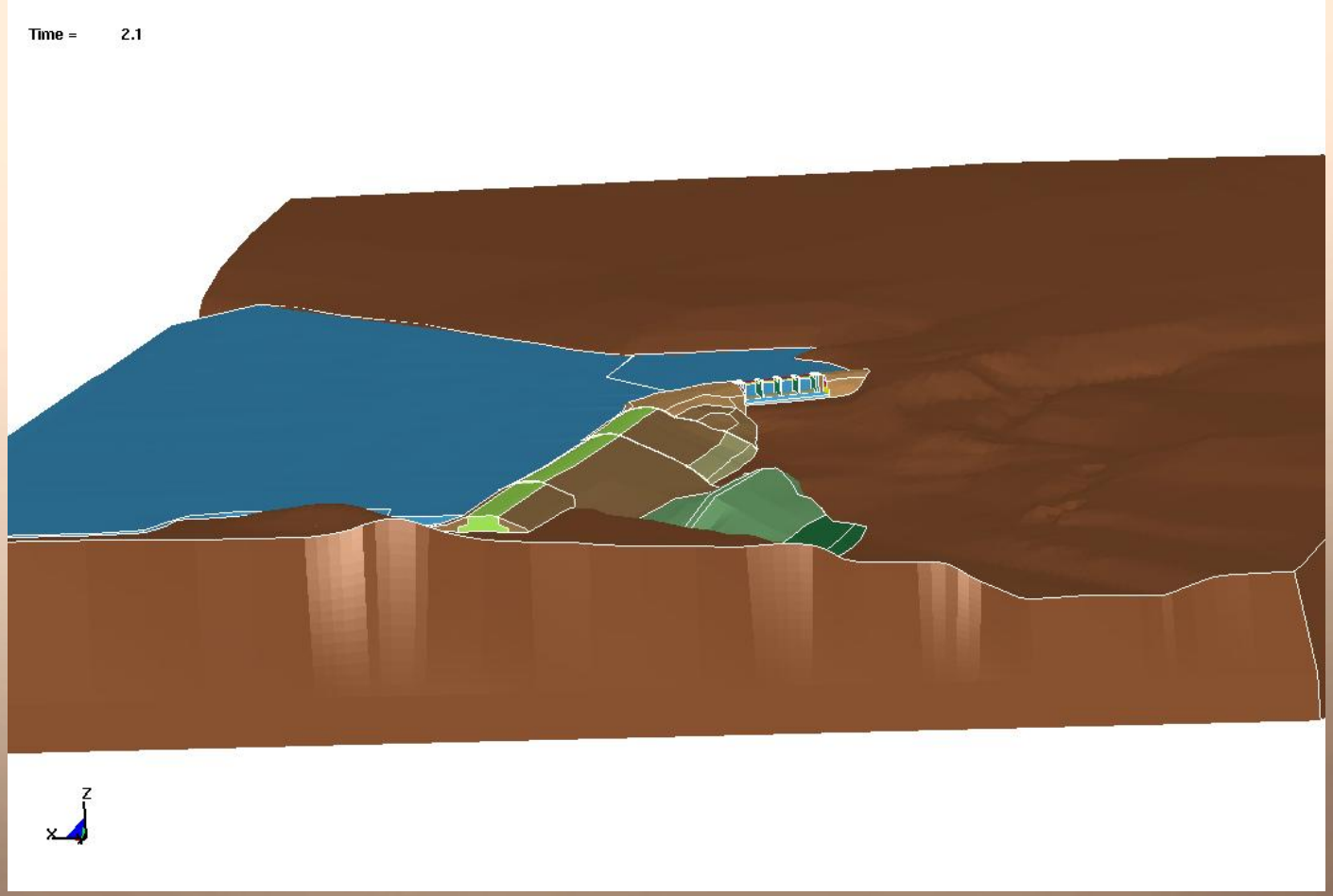
Finite Element Model of Soil-Wall System

- Soil-structure interaction can be evaluated with finite element model of wall and backfill
- Soil is modeled with non-linear properties so that soil can yield
- Contact surfaces are provided between the wall and the soil backfill
- Approach has been used for a number of Reclamation spillway crest structures

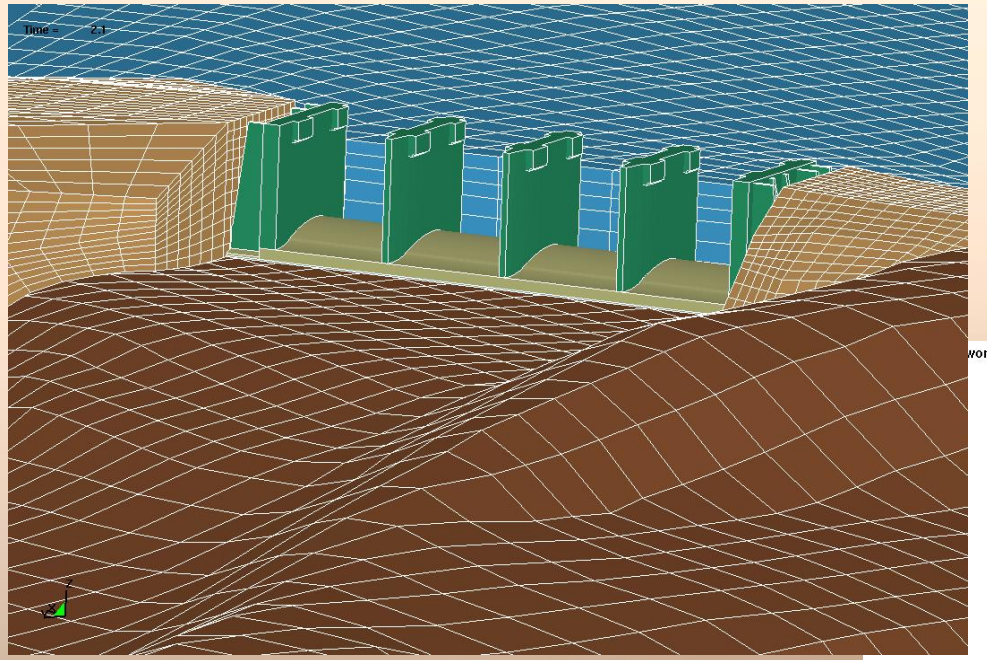
Finite Element Model of Soil-Wall System

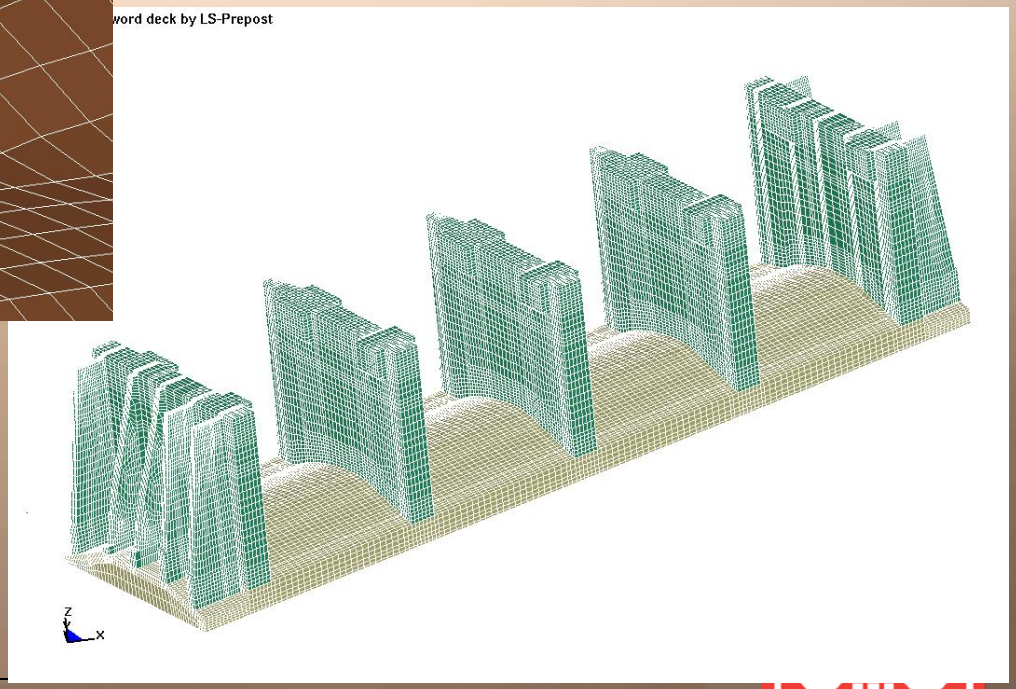
- At Pineview Dam, resulting earth pressures fell in between Mononobe-Okabe and Wood's solution
- At Green Mountain Dam, results approached passive condition
- Significant amount of effort is needed to verify and test model
- Sensitivity analyses are critical to evaluate changes in soil properties and models, boundary conditions and methods of applying loads

Bradbury Dam Model



Spillway Crest Structure Model





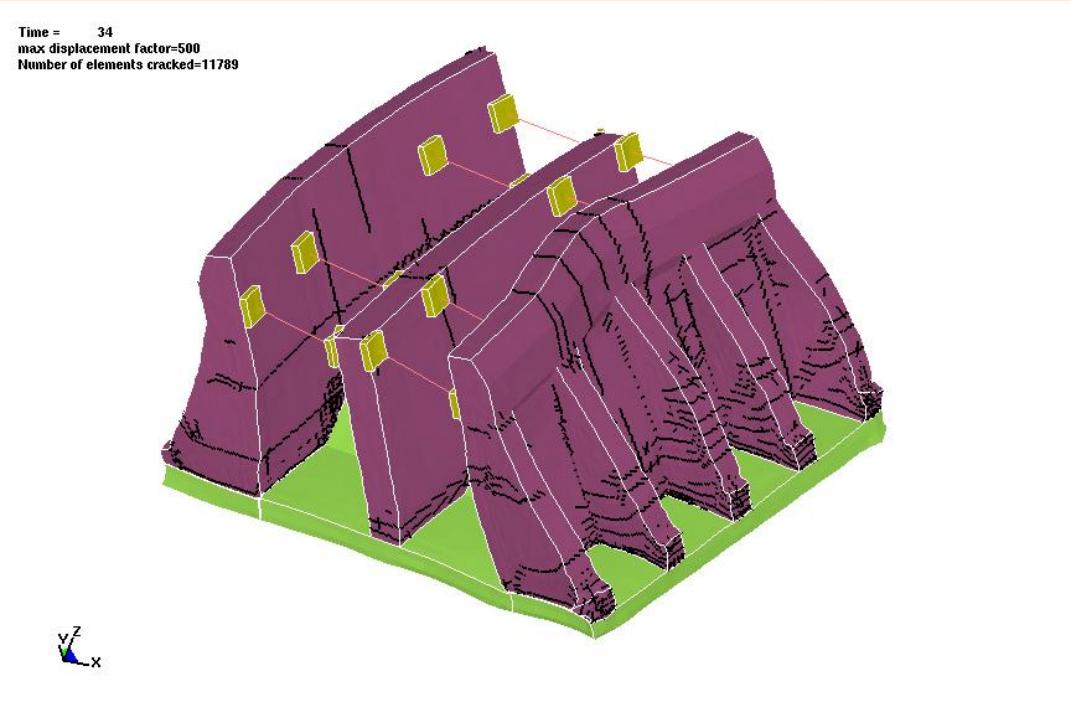
Scoggins Dam Model

Bradbury Dam and Scoggins Dam Crest Structure Analysis

- Full embankment dam and foundation included in model – plastic kinematic material models
- Reinforced concrete modeled with non-linear material properties
- Concrete was allowed to crack and reinforcement allowed to yield
- Crest structure was evaluated for shear and moment failures; non-linear deflections considered

Spillway Wall and Pier Failure

- Full nonlinear results – concrete cracking, reinforcing yielding



- Walls and piers crack and are damaged, but remain standing

Results from Bradbury and Scoggins Crest Structure Models

- Crest structure walls and piers will crack and some of the reinforcement will yield but loads are redistributed
- Walls and piers are not expected to fail or deform excessively
- Spillway gates can withstand up to 50k earthquake pseudo-static loads with no distress
- Earth pressure loads are between what would be provided by Mononobe-Okabe and Woods

Research by Sitar and Al-Atik

- Centrifuge modeling and numerical modeling of U-shaped retaining walls were conducted with sand backfill
- Models were subjected to ground motions and earth pressures and moments in the wall were measured
- Good historic performance of reinforced concrete walls during earthquakes was also documented

Findings – Sitar and Al-Atik

- Wall inertial moment contribution to total dynamic moments is substantial
- Wall inertial moments are generally in phase with dynamic wall movements
- Earth pressure moments are generally out of phase with dynamic wall moments
- Earth pressure distributions are triangular – same as static earth pressures
- Earth pressures on wall less than those predicted by Mononobe-Okabe

Reclamation Results vs. Sitar – Al-Atik Research

- While Sitar – Al-Atik research indicates that accepted methods may overstate earth pressure loads on walls, Reclamation studies of spillway crest structure walls indicates that earth pressures can be greater than accepted methods
- The primary differences between the Sitar - Al-Atik results and the Reclamation FLAC and LS-DYNA results are:
  - Various geometries and backfill conditions for crest structure walls
  - Various foundation conditions
- Reclamation has modeled the centrifuge experiments with LS-DYNA and has had some success duplicating the results
- Pursuing other physical model studies

Pier Exercise

- Consider a spillway with concrete piers that are 5-feet thick and 40-feet high. Calculate the shear stresses at the base of the pier in the cross-canyon direction only, for the earthquakes described in Table V-6-4. Assume that the ultimate shear capacity of the spillway piers is 200 lb/in^2. Assume that there are no bridges that will load the pier. Based on a comparison of the shear stress at the base of the pier to the shear capacity of the pier concrete, estimate the probability that the cross-canyon shear capacity will be exceeded for the 1000-, 5000-, 10,000-, and 50,000-year earthquake. (provides information that would be used in middle of event tree for shear node)

Pier Exercise

Table V-6-4 - Spillway Pier Analysis – Earthquake Loads	
Recurrence Interval, yr	Peak Horizontal Ground Acceleration
1000	0.2g
5,000	0.4g
10,000	0.5g
50,000	0.6g

Pier Exercise Solution

- The magnification factor for the pier is determined from Figure V-6-8, using the period of the structure calculated below and 2 percent damping:
- $T_s = 0.000643 \times H^2/B = 0.000643 \times 1600/5 = 0.21$
- From Figure V-6-8, the magnification factor is 2.1. Figure 1 depicts the inertial loading at the base of the pier for the 0.2g load case. The shear at the base of the pier is calculated below.
- $V = ½(150\ lb/ft^2 + 315\ lb/ft^2) \times 40 = 9300\ lb$
- The shear stress at the base of the pier is calculated below:
- $v = 9300/(60 \times 12) = 13\ lb/in^2$

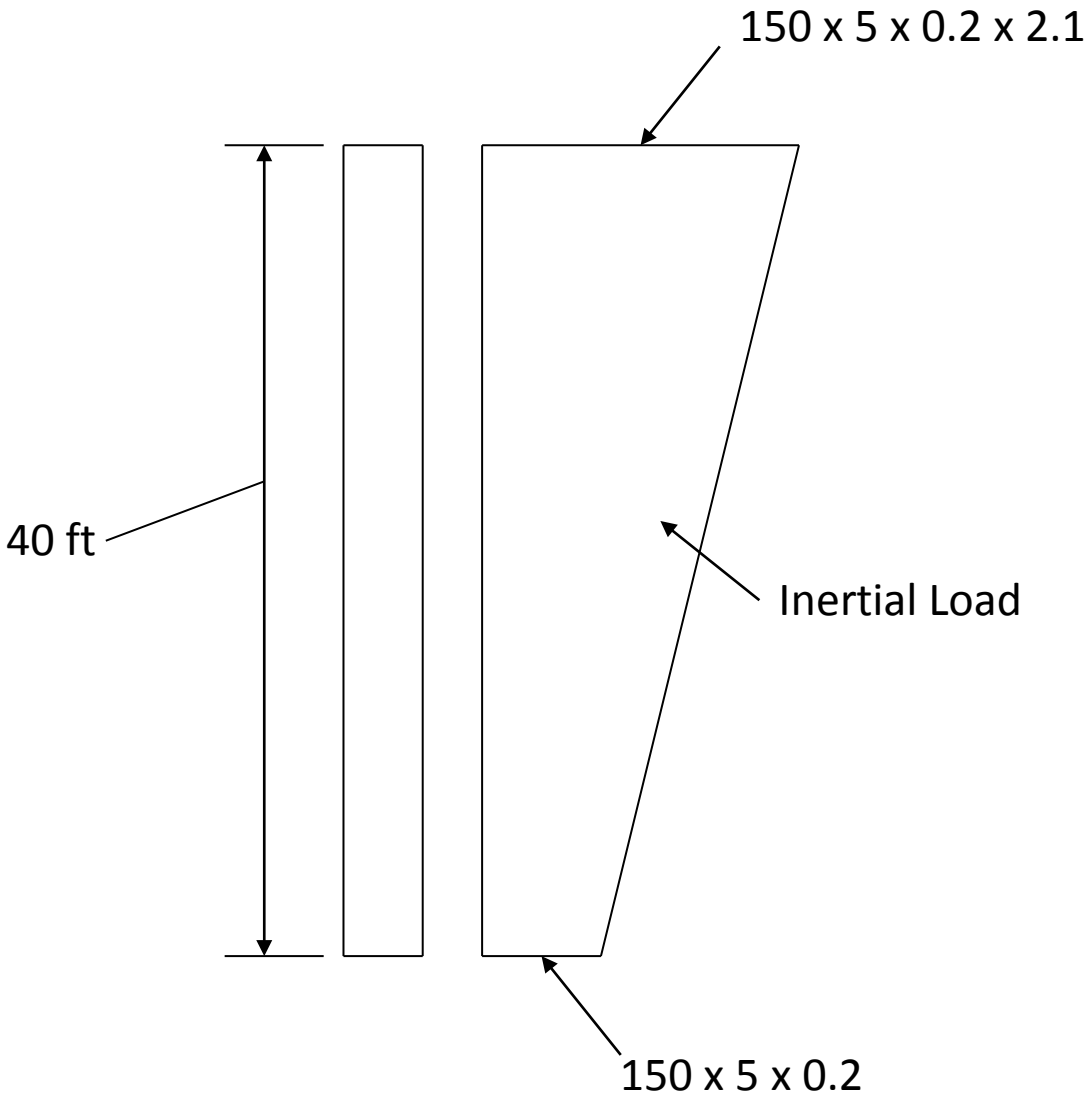


Figure 1

Shear Stresses at Base of Pier			
Recurrence Interval, yr	Peak Horizontal Ground Acceleration	Total Shear Force at Base of Pier	Shear Stress at Base of Pier
1000	0.2g	9300 lb	13 lb/in^2
5000	0.4g	18,600 lb	26 lb/in^2
10,000	0.5g	23,250 lb	33 lb/in^2
50,000	0.6g	27,900 lb	39 lb/in^2

Since the shear stresses are very low for all load cases and well below the stated shear capacity of 200 lb/in^2, the estimates for shear capacity being exceeded would all be very low (0.001).

Wall Exercise

- Consider a spillway with crest structure cantilever walls that are 2-feet thick at the base and 40-feet high. Calculate the shear stresses at the base of the wall for the earthquakes described in Table V-7-2, using Mononobe-Okabe for active earth pressures and assuming cohesionless backfill with a friction angle of 30° and a density of 120 lb/ft^3. Assume that the backfill is at the top of the walls and that the backfill surface is horizontal. Assume that the angle of interface friction (δ) is 15°. Assume that the shear capacity of the spillway walls is 200 lb/in^2, and that it remains constant for all loading conditions. Based on a comparison of the shear stress at the base of the wall to the shear capacity of the wall concrete, estimate the probability that the shear capacity will be exceeded for the 1000-, 5000-, 10,000-, and 50,000-year earthquake. Assume that there is no vertical component of the ground motions.

Wall Exercise

Table V-7-2 – Spillway Wall Analysis – Earthquake Loads	
Recurrence Interval, yr	Peak Horizontal Ground Acceleration
1000	0.1g
5,000	0.2g
10,000	0.3g
50,000	0.4g

Wall Exercise Solution

- The active earth pressure coefficients were obtained from Figure 18-6 for a friction angle of 30°. The active earth pressure coefficients are shown in Table 1. The total earth pressure (both static and dynamic) were calculated using the following equation:
- $P_{AE} = K_{AE}[1/2(\gamma_t(1 - k_v))H^2$
- The vertical ground acceleration was assumed to be 0. The shear at the base of the wall is calculated below.
- $V = K_{AE}[1/2(\gamma_t)]H^2 = K_{AE}[1/2(120)]40^2$
- The shear stress at the base of the wall is calculated below for the 1000-yr earthquake:
- $v = V/(24 \times 12) = 33{,}600/288 = 117\ lb/in^2$
- The following table summarizes the other load case results:

Table 1 - Shear Stresses at Base of Wall				
Recurrence Interval, yr	Peak Horizontal Ground Acceleration	K_{AE}	Total Earth Pressure Force	Shear Stress at Wall Base
1000	0.1g	0.35	33,600 lb	117
5000	0.2g	0.43	41,280 lb	143
10,000	0.3g	0.55	52,800 lb	184
50,000	0.4g	0.70	67,200 lb	233

Based on the calculated shear stresses above and the shear capacity of 200 lb/in^2, the following estimates that the shear capacity will be exceeded were made:

1000 yr – 0.001
5000 yr – 0.001
10,000 yr – 0.1
50,000 yr - 0.99