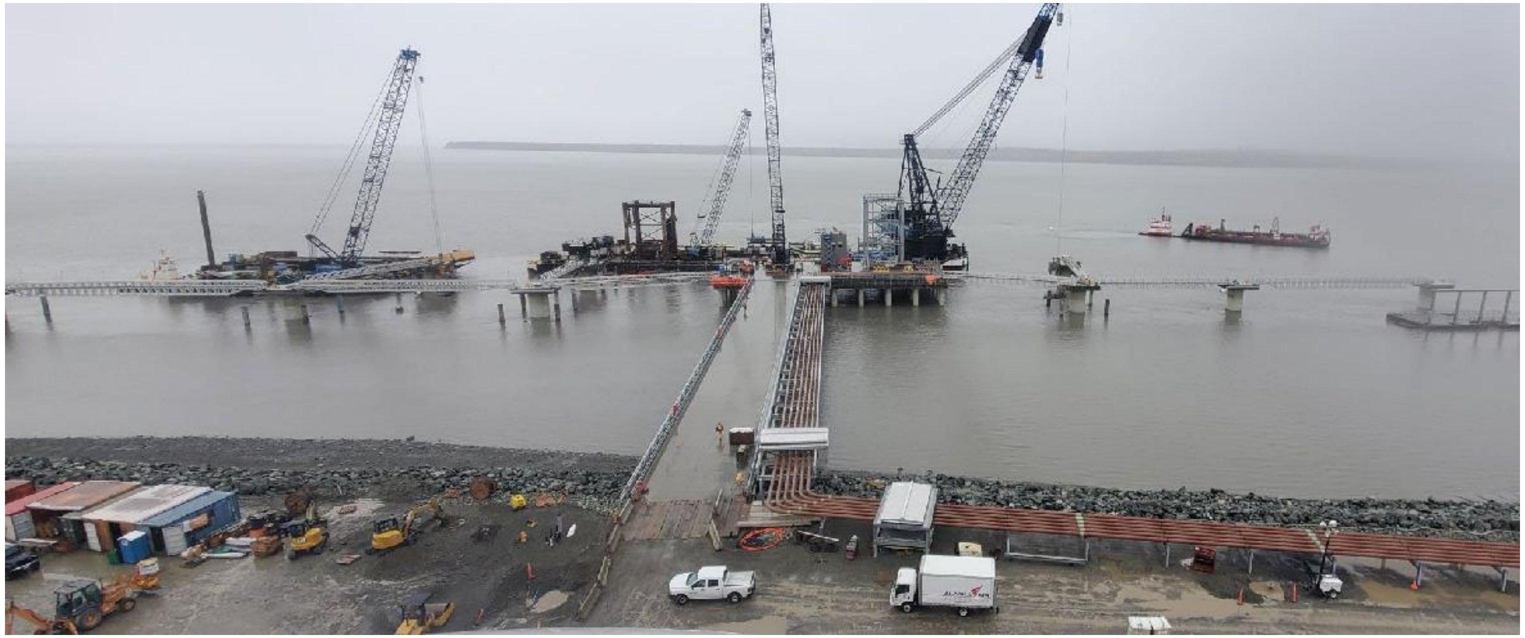
# Port of Alaska Update
# SEAAK - May 17, 2023


# PROGRAM UPDATE

- **PHASE 1** Petroleum and Cement Terminal **complete**
- **PHASE 2A** underway:
  - NES1 and new Administration Building
- **PHASE 2B** in design and permitting:
  - Cargo Docks Replacement, RORO/LOLO Container Terminals
- Future **PHASES 3, 4,** and **5**:
  - NES2, Petroleum Terminal, and Remaining Demolition

# PHASE 1 complete

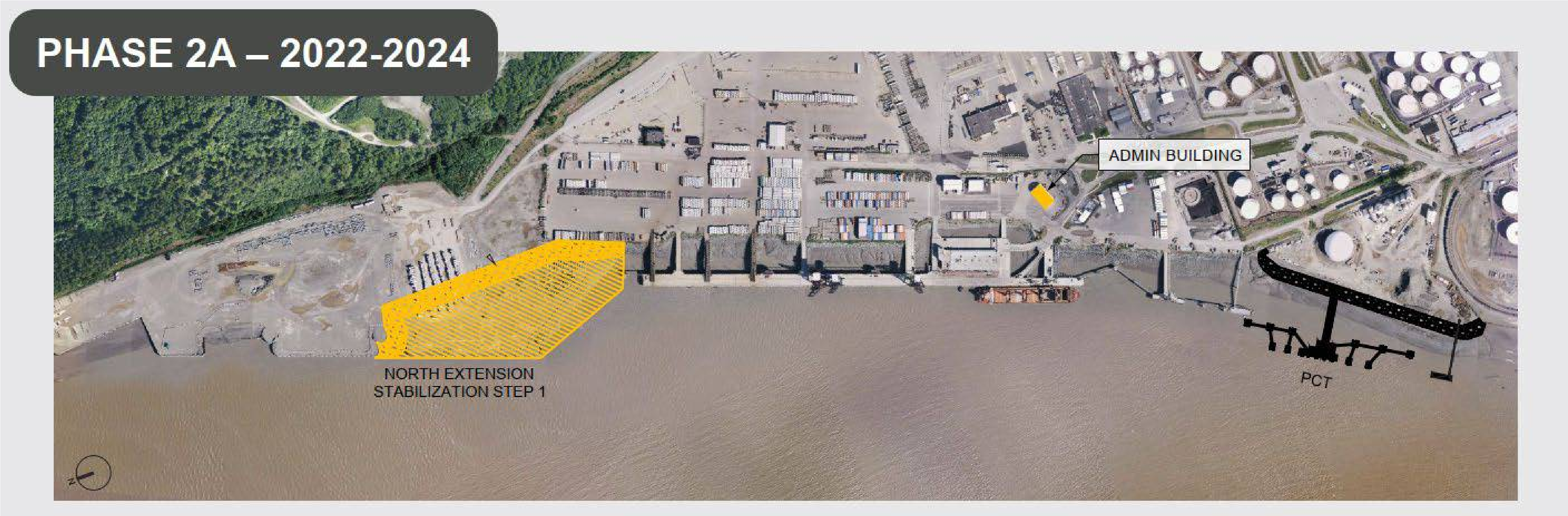


# Phase 1 complete

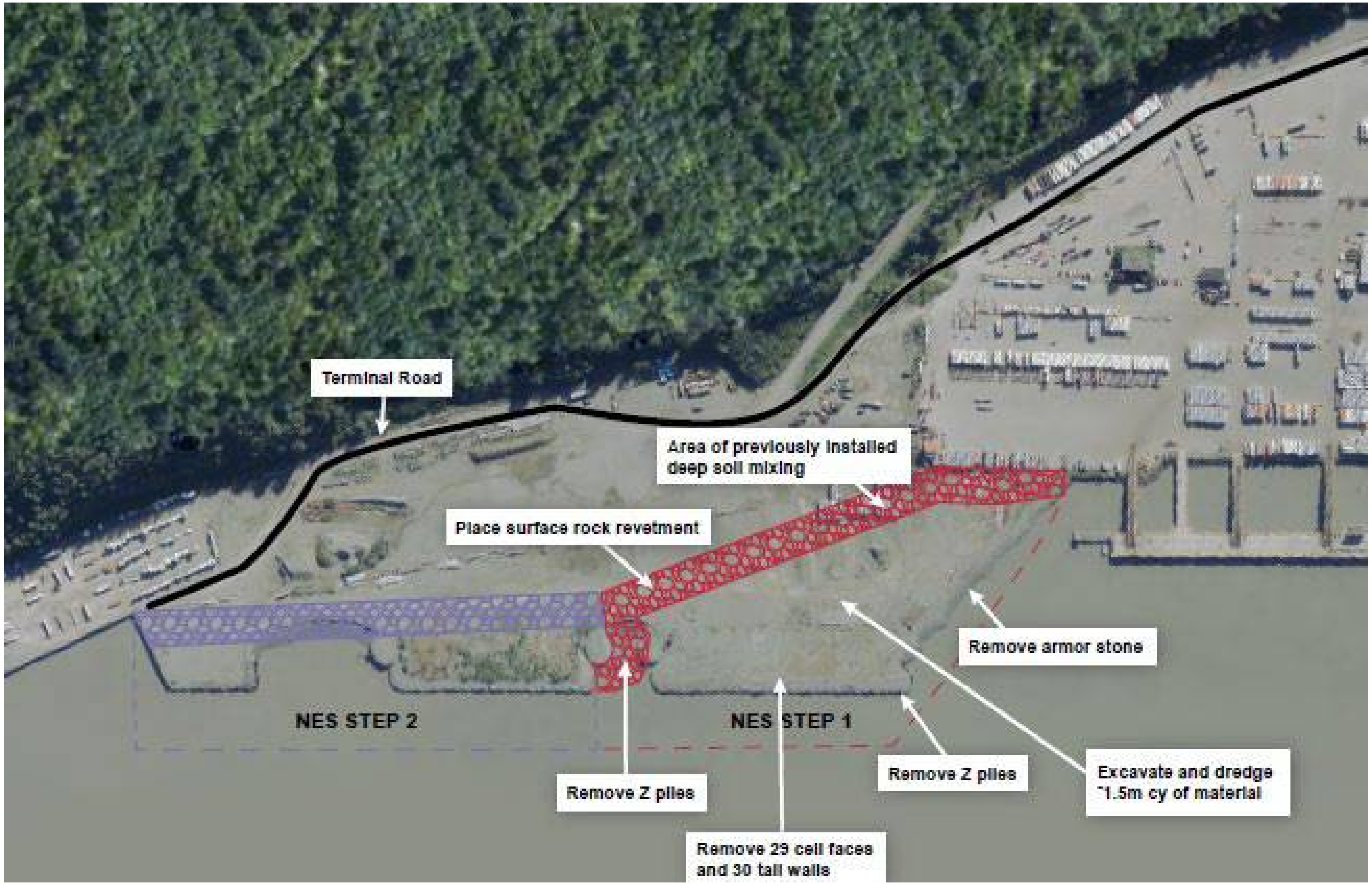
# PCT Fact Sheet

- Four primary contracts from 2018 to 2022
- Total cost approximately $220 million
- 140,000 manhours in 2021 alone
- 71 48-in-diameter piles, 180 feet long
- 9 12-ft-diameter monopiles
- Prime contractor for dock construction: Pacific Pile & Marine

# North Extension Stabilization Step 1 (NES1)



# North Extension Stabilization Step 1 (NES1)



# NES1 Design-Build Contract

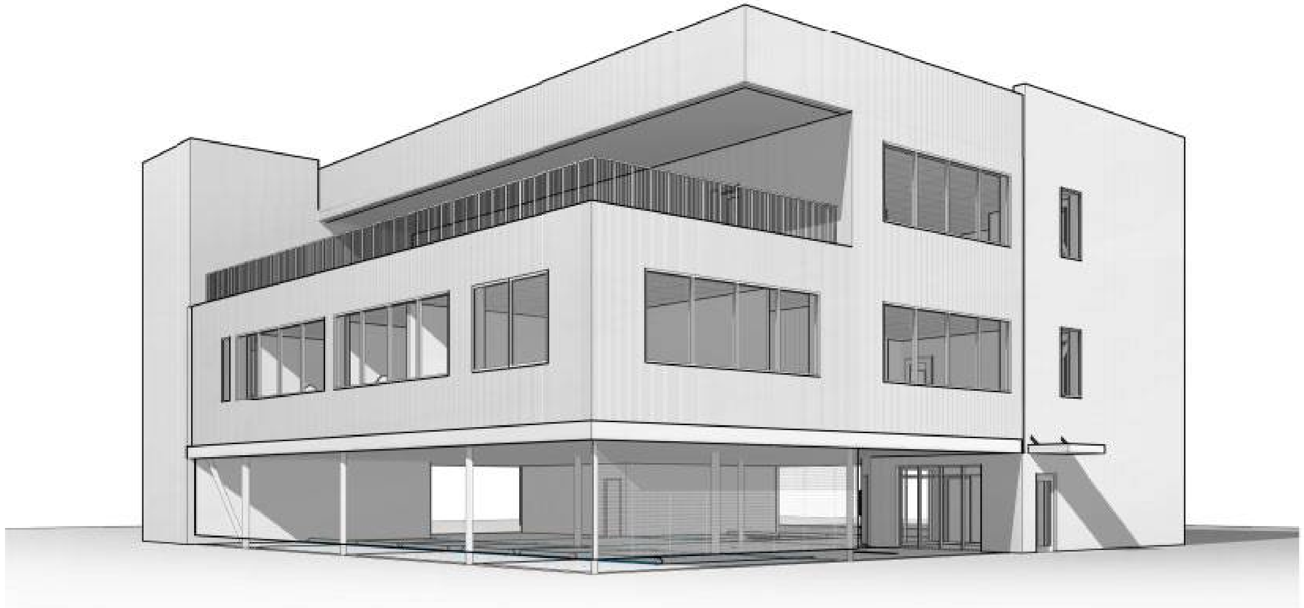
- Recommendation to Award made to MOA Assembly
- Total contract value: $97 million plus contingency
- NTP expected in December
- Prime contractor: Manson Construction Co.

# Subcontractors on NES1

- Granite Construction
- Condon-Johnson and Associates
- Edge Survey and Design LLC
- WSP
- Farwest Fabrication
- 61 North Consulting

# New Administration Building

- Design-Build Contract
- Contract value: $9.3 million plus contingency
- Construction completion: Spring 2024
- Prime contractor: STG Pacific



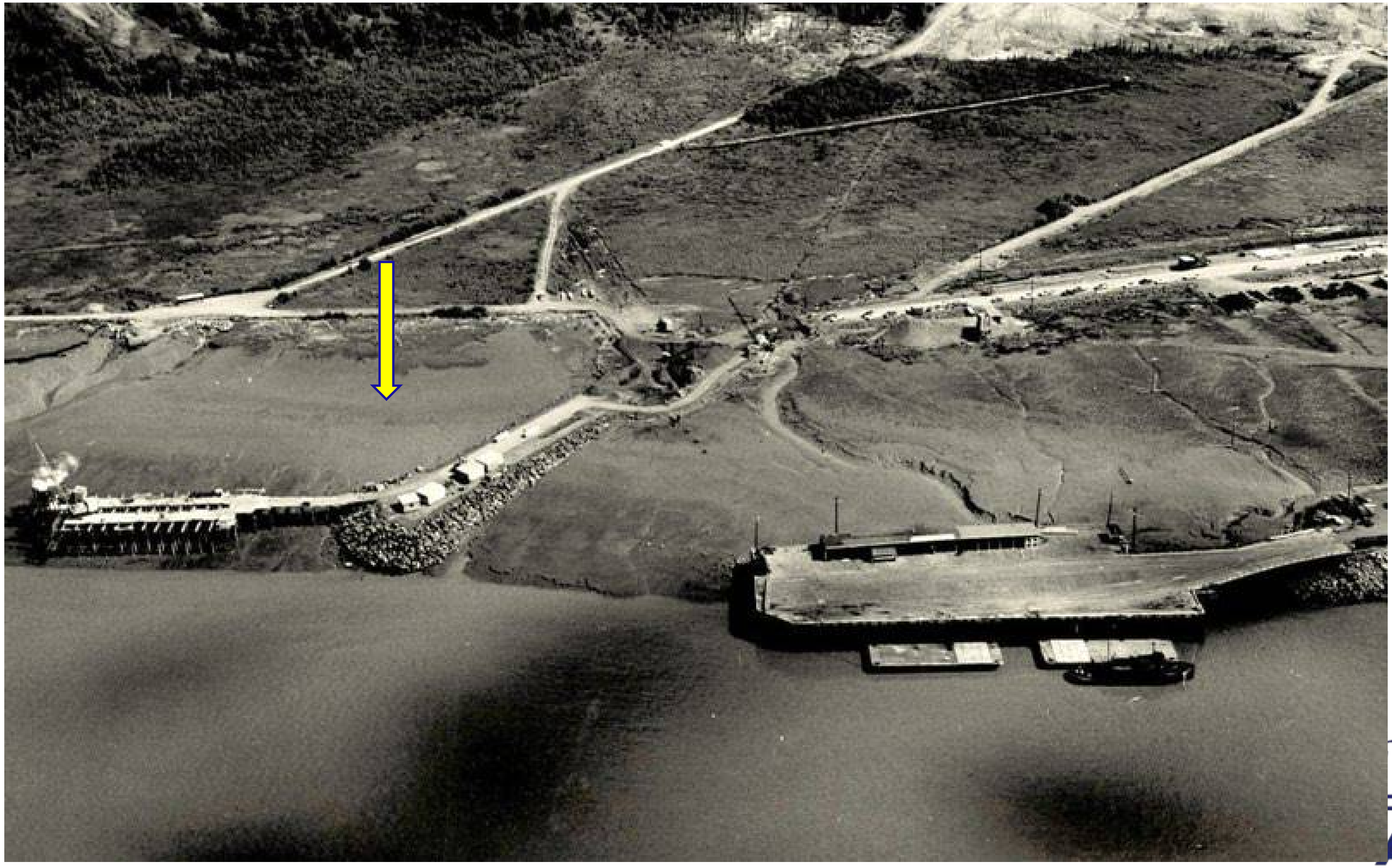
# Subcontractors on Admin Building

- Design, Engineering and Surveying:
  - RIM Architects
  - Golder
  - BBFM
  - Lounsbury Inc.
- Construction:
  - TK Elevator
  - Strata Deep Constructors
  - Haakenson Electrical
  - Klebs Heating
  - Whalen Construction

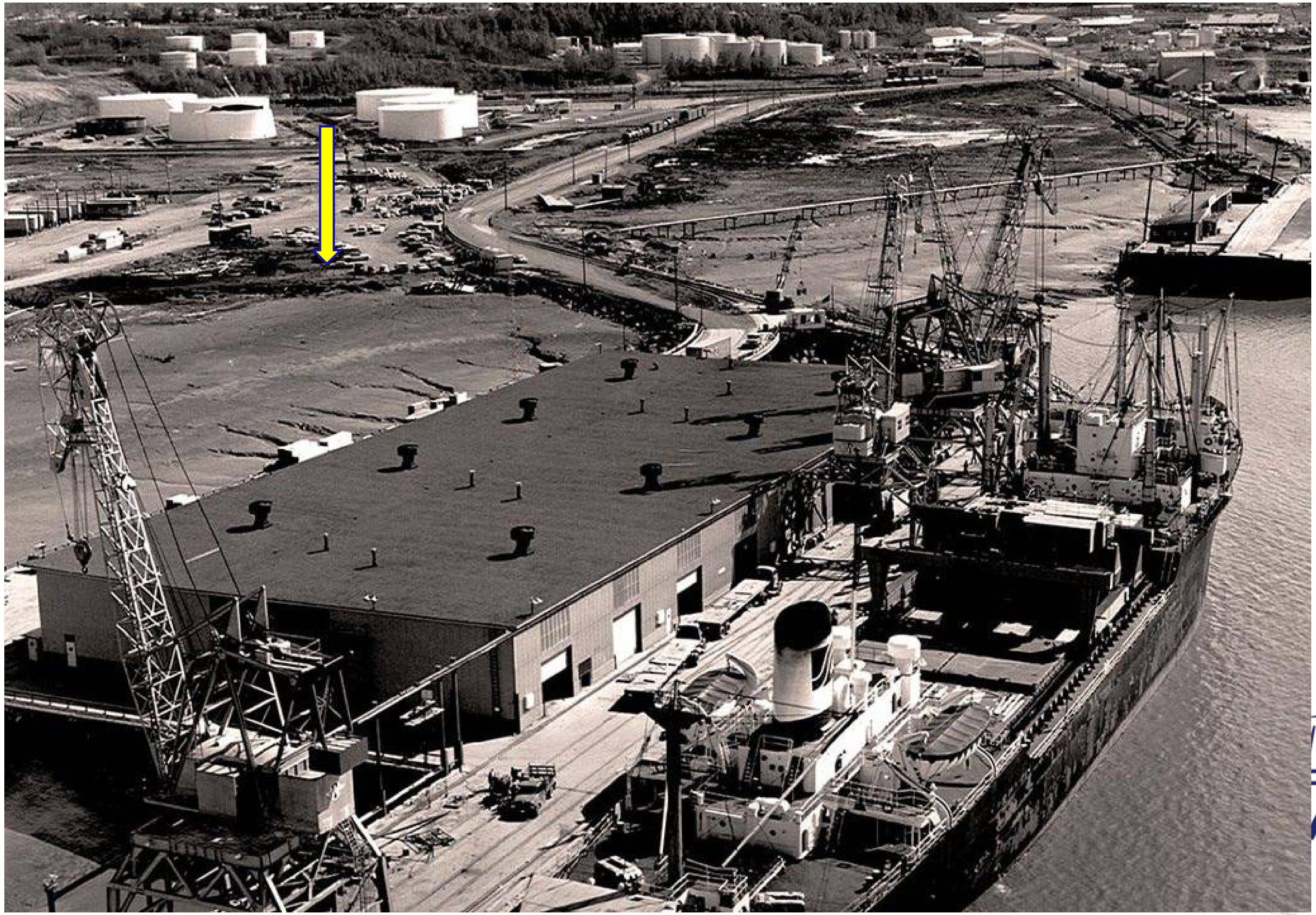
# Helical pile damaged by debris.

# Concrete debris at the Admin Building site

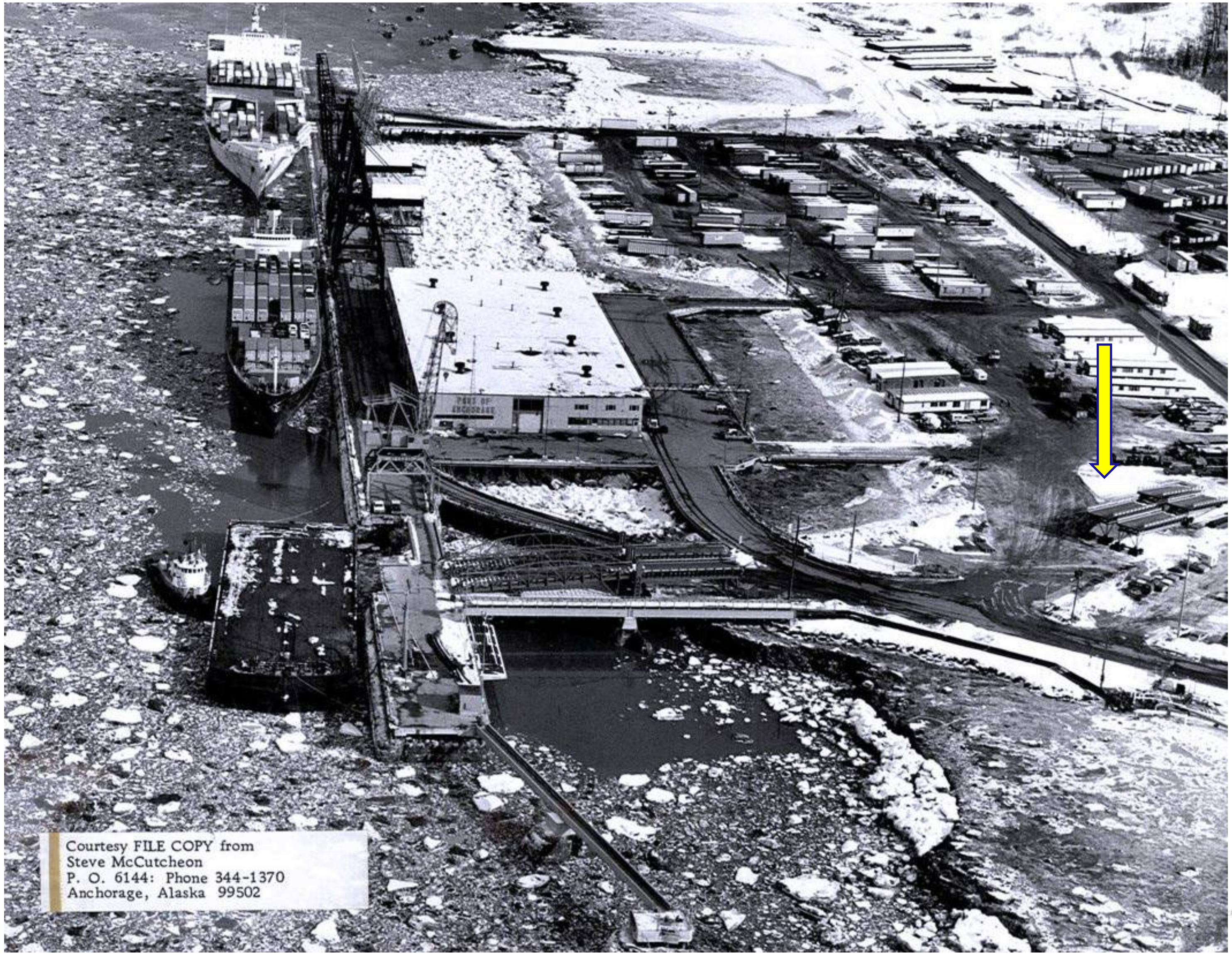
# Admin Building site 1959 – No Fill



# Admin Building site 1964 – Partial Fill



# Admin Building site late 1960s – Filled







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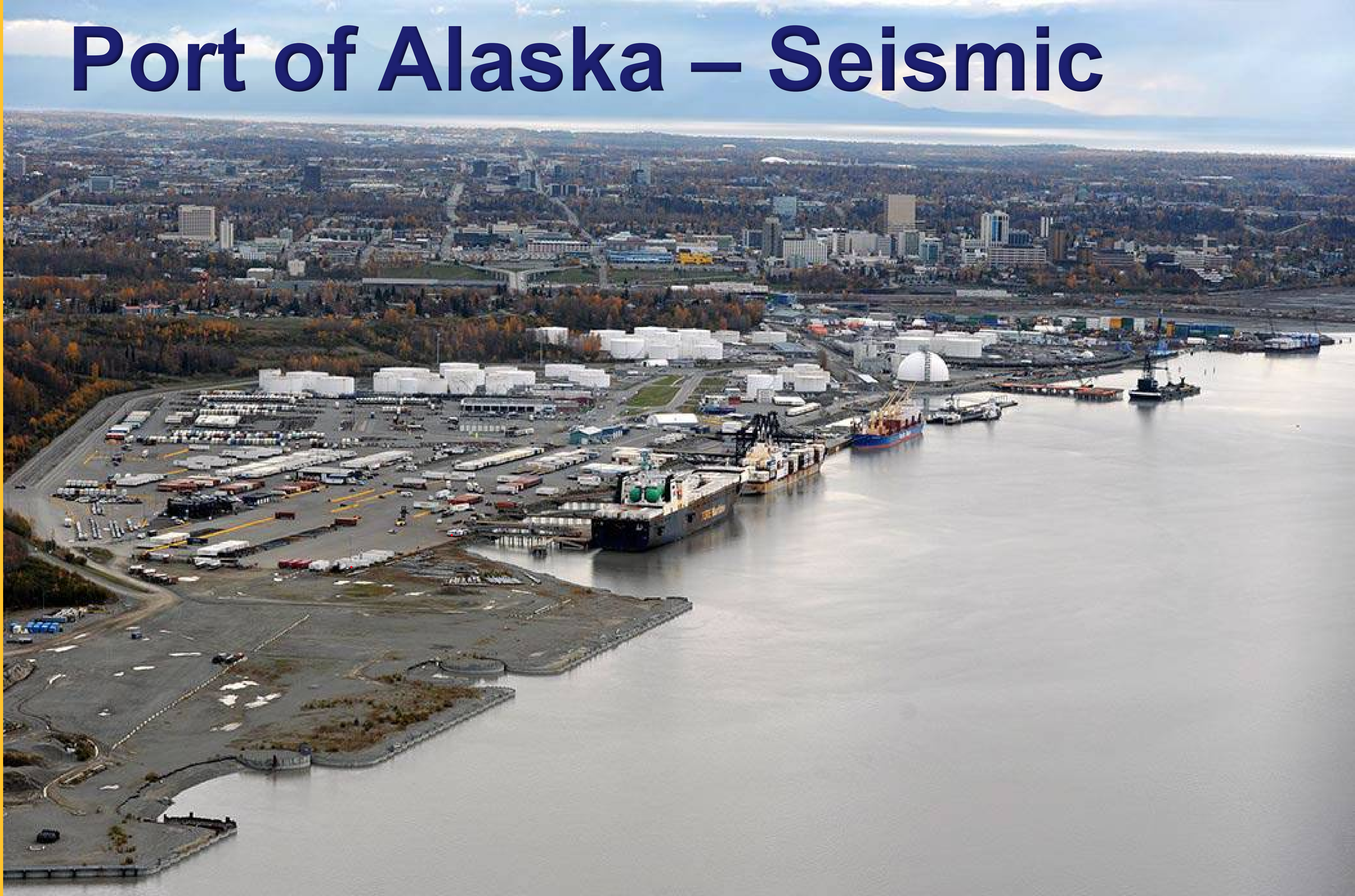
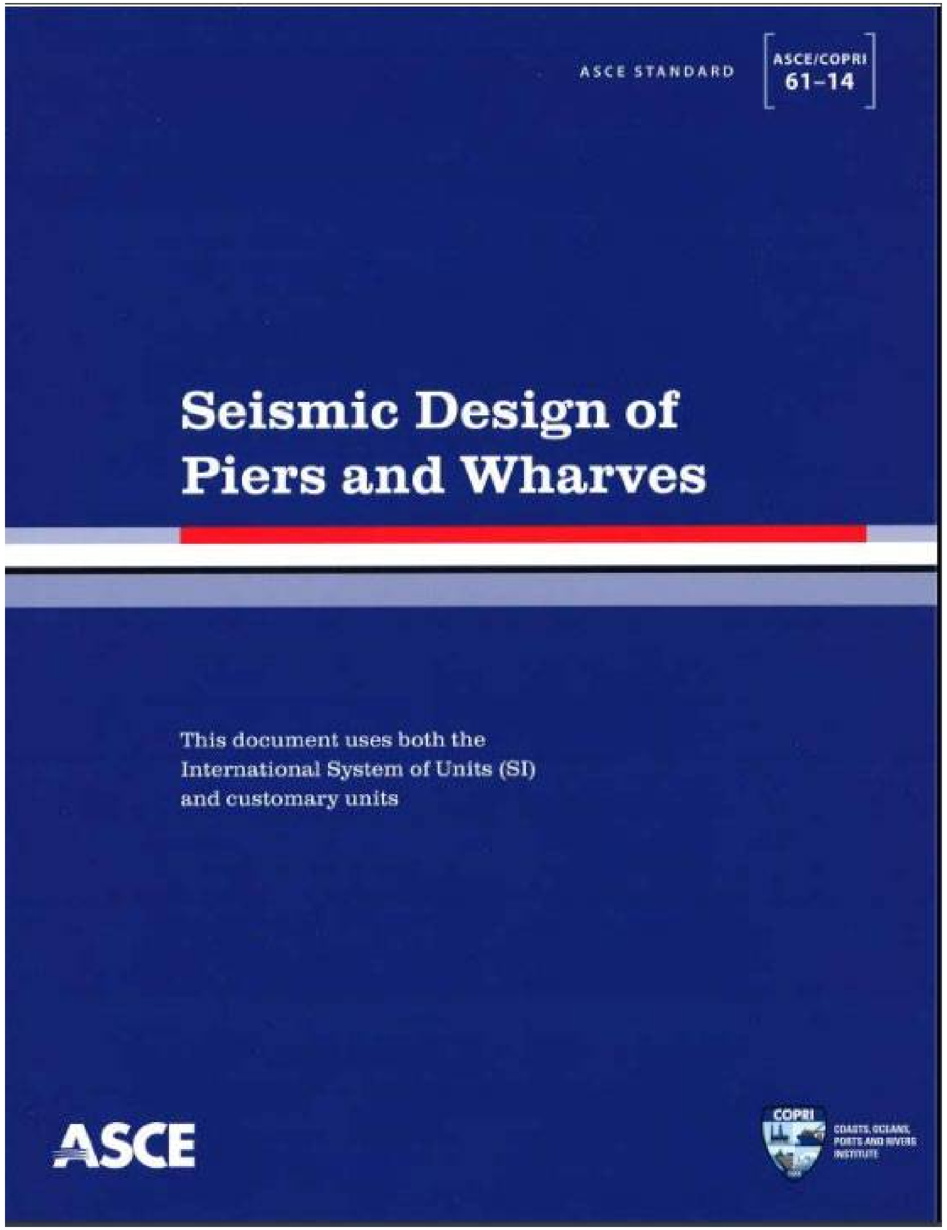
PORT of ALASKA IN ANCHORAGE

# Port of Alaska – Seismic



# ASCE 61 -14 Performance Requirements (Code)



# ASCE 61 -14 Performance Requirements (Code)

| DESIGN CLASSIFICATION | SEISMIC HAZARD LEVEL AND PERFORMANCE LEVEL | | | | | |
|---|---|---|---|---|---|---|
| | Operating Level Earthquake (OLE) | | Contingency Level Earthquake (CLE) | | Design Earthquake (DE) | |
| | Ground Motion Probability of Exceedance | Performance Level | Ground Motion Probability of Exceedance | Performance Level | Seismic Hazard Level | Performance Level |
| HIGH | 50% in 50 years (72-year return period) | Minimal Damage | 10% in 50 years (475-year return period) | Controlled and Repairable Damage | as per ASCE 7 | Life-Safety Protection |
| MODERATE | n/a | n/a | 20% in 50 years (224-year return period) | Controlled and Repairable Damage | as per ASCE 7 | Life-Safety Protection |
| LOW* | n/a | n/a | n/a | n/a | as per ASCE 7 | Life-Safety Protection |

# GAC 2014 letter

From September 23, 2014 GAC letter:

We agree with the Port that, at a minimum, one container dock and one POL dock should be designed for "minimal damage" at the CLE ground motions (rather than "controlled and repairable damage" as the CLE motions referenced in the code), and "controlled and repairable damage" at the DE ground motions. These structures will be referred to as the "seismic berths" in this letter.

# GAC 2014 letter

From September 23, 2014 GAC letter:

"Controlled and Repairable Damage" by definition implies there could be loss of serviceability for "several months". That time frame is likely to be too long to supply 80% to 90% of the goods for the entire State, particularly in winter conditions. The commission advises that the definition of "controlled and repairable damage" should be adjusted to mean damage which is feasibly repairable within several days to one week of the seismic event. We advise that contingencies, plans, and materials be included in the design for repairs in the event of a Design Earthquake to reduce response time.

Comment: The interpretation of this is not in line with the intent of the ASCE 61 committee. The intent of the "Controlled and Repairable Damage" state is to maintain some level of serviceability.

# 2014 GAC Recommended Performance Requirements Minimal Damage in 2/3 MCE

effective

| DESIGN CLASSIFICATION | SEISMIC HAZARD LEVEL AND PERFORMANCE LEVEL | | | | | |
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# The “feasibly repairable in one-week” criteria causes problems :

- Engineering Design Parameter or a Goal?
- There is no way for the design team to precisely control repair timeframe. (What if you have to go out to bid?)
- Damage does not necessarily equal out of service.

# ASCE 61-23 Suggested Language- Code

Performance is classified as “controlled and repairable damage” when (a) the structure responds in a controlled and ductile manner, experiencing limited inelastic deformations to an extent such that structural repair is possible, (b) the deck does not experience significant damage and pile damage is limited to an extent that no local collapses occur, (c) the structure may experience a temporary reduction in serviceability until inspection, evaluation, and/or repairs are performed, but maintains some level of serviceability, (d) damage to ancillary structures does not cause significant risk to life safety, and (e) there is no loss of containment of materials in a manner that would pose an immediate and direct public hazard.

# ASCE 61-23 Suggested Language-Commentary

It is important to recognize damage in this performance category typically will not result in a complete loss of service. For example, spalling of the concrete cover at the pile to deck or pile to cap interface is expected. This loss of concrete cover may expose the underlying steel reinforcing to the elements. This results in the risk of corrosion over time. Repair is therefore required. However, this does not equate to an immediate complete loss of serviceability to the facility.

# Revised Performance Requirements (ASCE 61-23)

| DESIGN CLASSIFICATION | SEISMIC HAZARD LEVEL AND PERFORMANCE LEVEL | | | | | |
|---|---|---|---|---|---|---|
| | Operating Level Earthquake (OLE) | | Contingency Level Earthquake (CLE) | | Design Earthquake (DE) | |
| | Ground Motion Probability of Exceedance | Performance Level | Ground Motion Probability of Exceedance | Performance Level | Seismic Hazard Level | Performance Level |
| HIGH | 50% in 50 years (72-year return period) | Minimal Damage | 10% in 50 years (475-year return period) | Controlled and Repairable Damage | 5% in 50 years (975-year Return Period) | Life-Safety Protection |
| MODERATE | n/a | n/a | 20% in 50 years (224-year return period) | Controlled and Repairable Damage | as per ASCE 7 | Life-Safety Protection |
| LOW* | n/a | n/a | n/a | n/a | as per ASCE 7 | Life-Safety Protection |

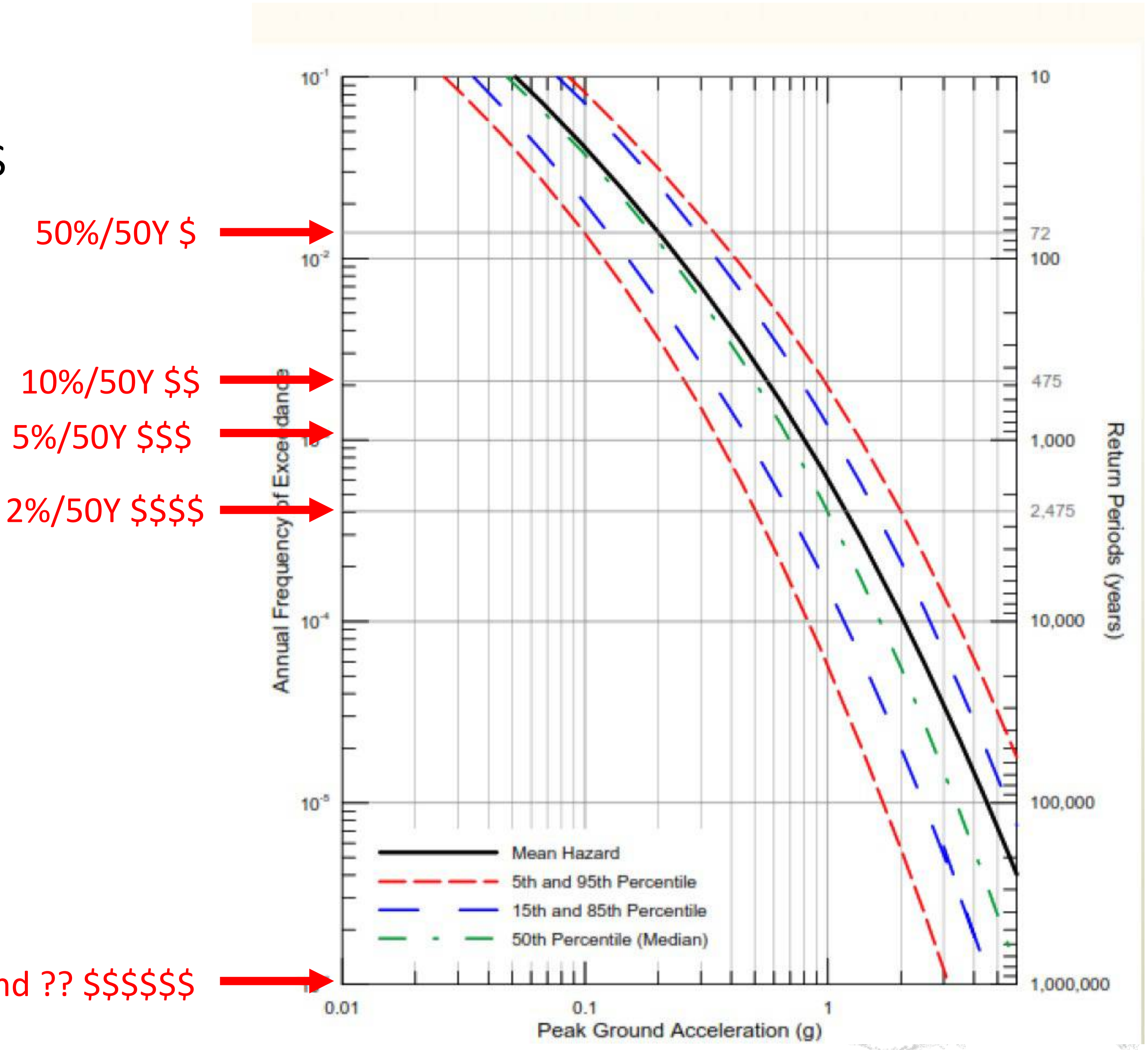
# Terminal Seismic Design

- "High" design classification.
- Performance criteria of "controlled and repairable damage" in the design event is one full level above national standards.
- We are one of the very few facilities in the world that have this high of a design standard.
- It is also true that we expect some damage but to remain in service following a design level event.

From Lettis Site Specific Hazard Analyses (horizontal accelerations top 30 meters)

It is possible to chase large infrequent earthquakes and resultant risk off into infinity.

What is a reasonable stopping point?



# POA Design Event Performance - Near Elastic

Near Elastic Design Return Period

Code →

GAC Clarification →

PCT →

| Return Period | Spectral Accelerations (g) | | | | | |
|---|---|---|---|---|---|---|
| | [illegible] | | [illegible] | | [illegible] | |
| | PGA | 1-second | PGA | 1-second | PGA | 1-second |
| 72-year | 0.201 | 0.135 | 0.148 | 0.064 | 0.16 | 0.1 |
| 475-year | 0.553 | 0.32 | 0.579 | 0.197 | 0.44 | 0.25 |
| 975-year | 0.791 | 0.469 | n/a | n/a | 0.61 | 0.38 |
| 2,475-year | 1.167 | 0.671 | 0.759 | 0.482 | 0.59 | 0.47 |

Approximate 4 times increase in accelerations between 72-year and 975-year return interval event!

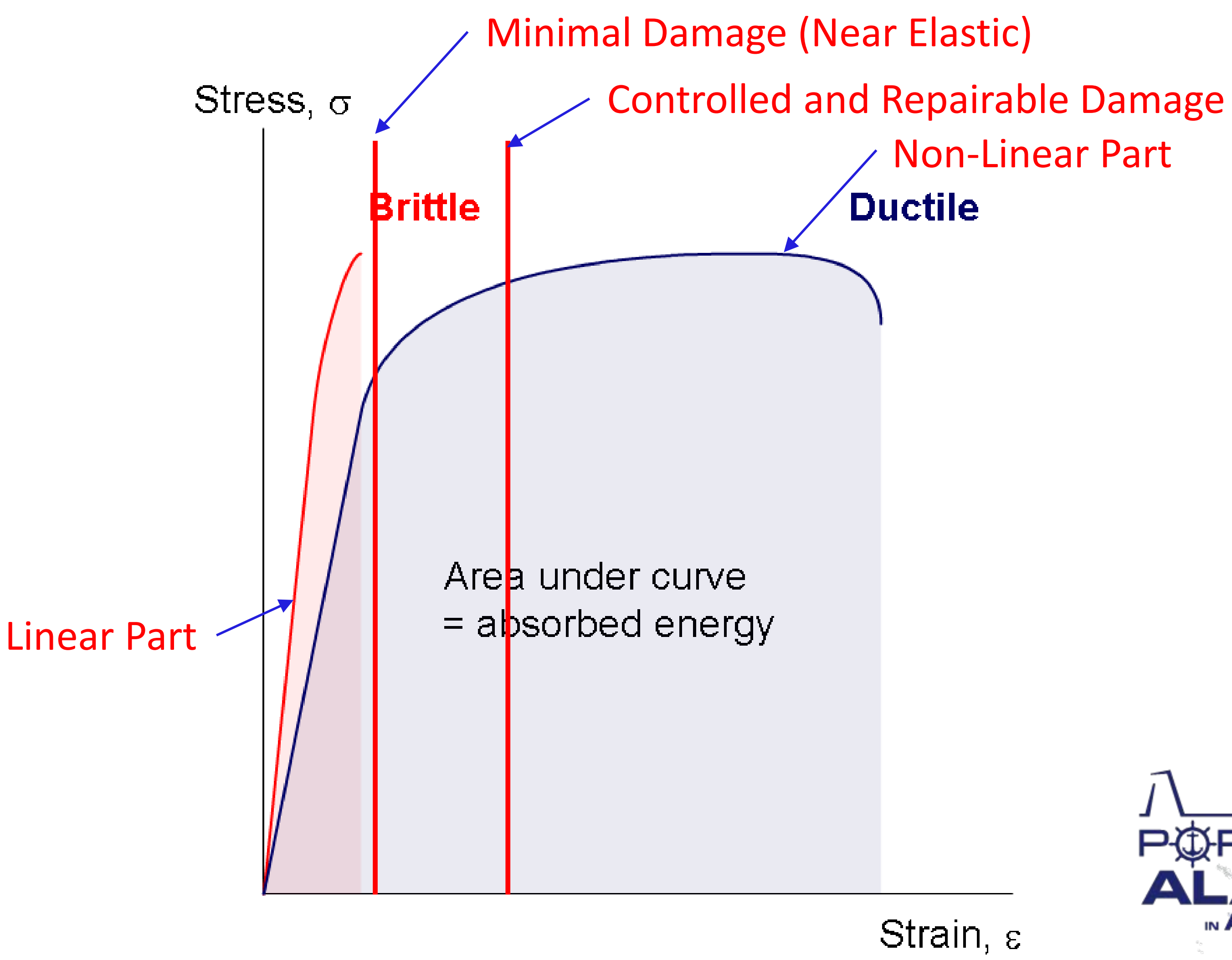
Near elastic performance for larger events has significant increase in design forces and cost!

# POA Costs Design Event Performance

| Performance Level in DE | Life Safety | Controlled and Repairable Damage | Minimal Damage |
|---|---|---|---|
| | | | |
| Cost per Square Foot | $500 | ? | $3,000 |
| | | | |
| Notes | Typical US West Coast Cost | No Data | One Data Point: PCT |

# Energy capacity = area under curve

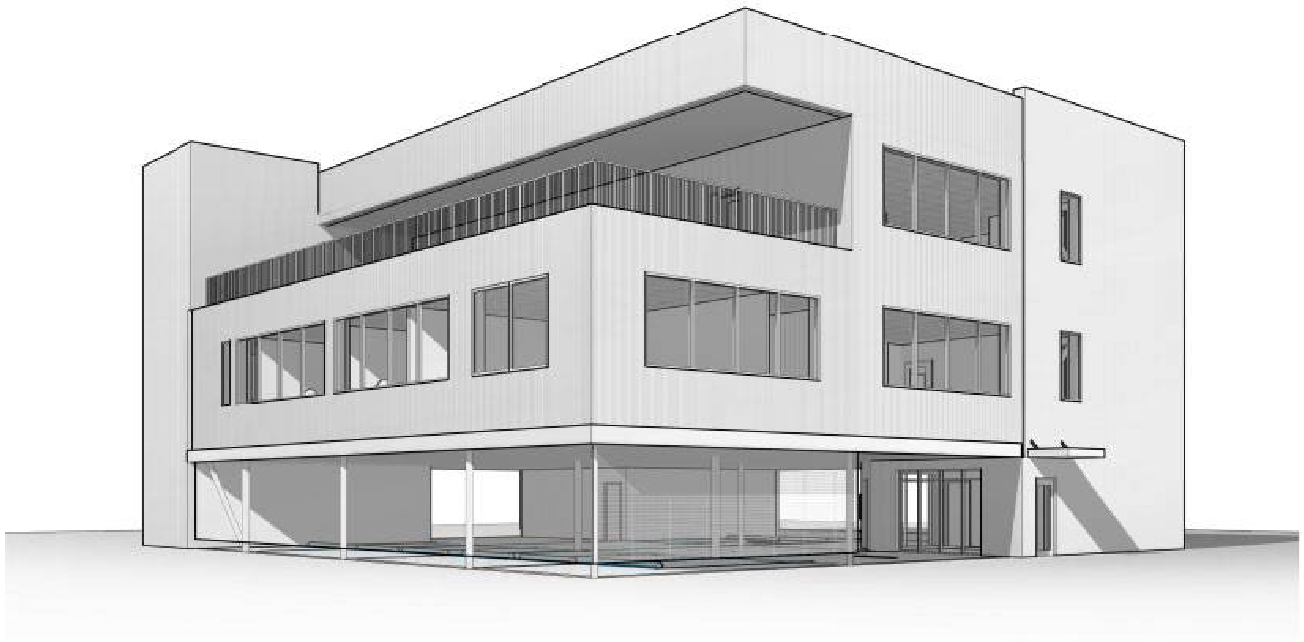
- 8 to 10 times yield capacity before collapse

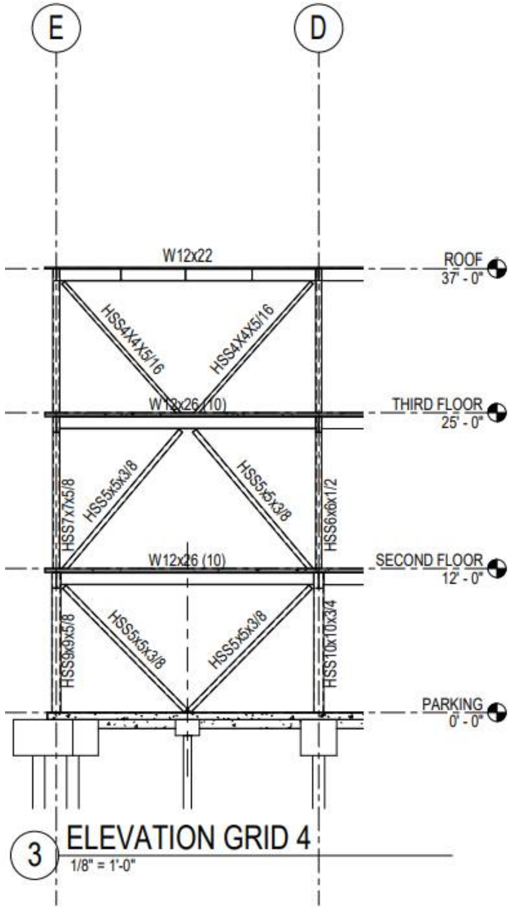


# Example Design Criteria new POA Admin Building (Force Based Design)

$10 million

Concentrically braced frame





# Example Design Criteria new POA Admin Building (Force Based Design)

- Risk Category II (Target Reliability, Conditional Probability of Failure in MCE 10%)
- Importance 1 (A factor to determine design loads)
- Sds 1.2; Sd1 0.771 (Short and long period adjusted design accelerations, mapped)
- Seismic Design Category D (High seismic vulnerability)
- Cs = 0.2; (Response / Equivalent lateral load factor)
- R=6.0; Concentrically braced frame (Response modification factor- ductility)
- Omega = 2.0 (Overstrength factor)

# Example new POA Admin Building

10% chance of conditional structural stability failure in MCE!

25% chance of conditional noncritical structural failure in MCE!

(Note 3 level ground motion similar to ASCE 61)

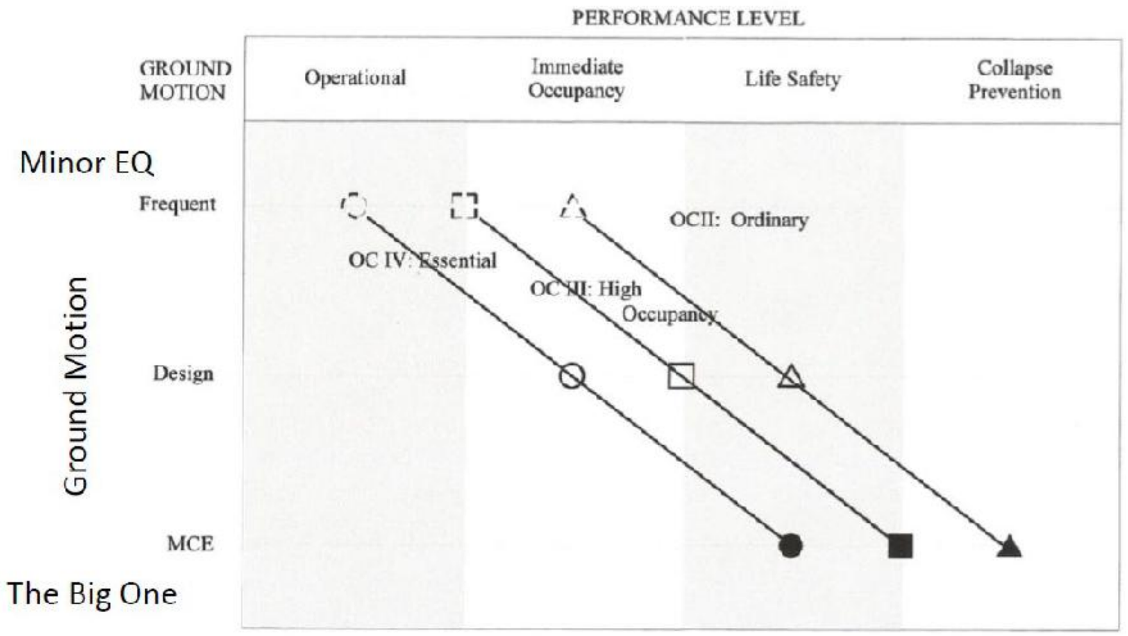



Figure C11.5-1 Expected performance as related to occupancy category (OC) and level of ground motion.

Expected Performance

# Seismic Design of Buildings - R and Omega

Base Shear

$$V = C_s W$$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$$

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} \quad \text{for} \quad T \leq T_L$$

Table 12.2-1. Design Coefficients and Factors for Seismic Force-Resisting Systems.

| Seismic Force-Resisting System | ASCE 7 Section Where Detailing Requirements Are Specified | R<br>Response Modification Coefficient, $R^a$ | Omega<br>Overstrength Factor, $\Omega_0^b$ | Deflection Amplification Factor, $C_d^c$ | Structural System Limitations Including Structural Height, $h_n$, Limits (ft)$^d$ — Seismic Design Category B | C | $D^e$ | $E^e$ | $F^f$ |
|---|---|---|---|---|---|---|---|---|---|
| **A. BEARING WALL SYSTEMS** | | | | | | | | | |
| 1. Special reinforced concrete shear walls | 14.2 | 5 | 2½ | 5 | NL | NL | 160 | 160 | 100 |
| 2. Reinforced concrete ductile coupled walls | 14.2 | 8 | 2½ | 8 | NL | NL | 160 | 160 | 100 |
| 3. Ordinary reinforced concrete shear walls | 14.2 | 4 | 2½ | 4 | NL | NL | NP | NP | NP |
| 4. Detailed plain concrete shear walls | 14.2 | 2 | 2½ | 2 | NL | NP | NP | NP | NP |
| 5. Ordinary plain concrete shear walls | 14.2 | 1½ | 2½ | 1½ | NL | NP | NP | NP | NP |
| 6. Intermediate precast shear walls | 14.2 | 4 | 2½ | 4 | NL | NL | 40 | 40 | 40 |
| 7. Ordinary precast shear walls | 14.2 | 3 | 2½ | 3 | NL | NP | NP | NP | NP |
| 8. Special reinforced masonry shear walls | 14.4 | 5 | 2½ | 3½ | NL | NL | 160 | 160 | 100 |
| 9. Intermediate reinforced masonry shear walls | 14.4 | 3½ | 2½ | 2¼ | NL | NL | NP | NP | NP |
| 10. Ordinary reinforced masonry shear walls | 14.4 | 2 | 2½ | 1¾ | NL | 160 | NP | NP | NP |
| 11. Detailed plain masonry shear walls | 14.4 | 2 | 2½ | 1¾ | NL | NP | NP | NP | NP |
| 12. Ordinary plain masonry shear walls | 14.4 | 1½ | 2½ | 1¼ | NL | NP | NP | NP | NP |
| 13. Prestressed masonry shear walls | 14.4 | 1½ | 2½ | 1¾ | NL | NP | NP | NP | NP |
| 14. Ordinary reinforced AAC masonry shear walls | 14.4 | 2 | 2½ | 2 | NL | 35 | NP | NP | NP |
| 15. Ordinary plain AAC masonry shear walls | 14.4 | 1½ | 2½ | 1½ | NL | NP | NP | NP | NP |
| 16. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance | 14.5 | 6½ | 3 | 4 | NL | NL | 65 | 65 | 65 |
| 17. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets | 14.1 | 6½ | 3 | 4 | NL | NL | 65 | 65 | 65 |
| 18. Light-frame walls with shear panels of all other materials | 14.1 and 14.5 | 2 | 2½ | 2 | NL | NL | 35 | NP | NP |
| 19. Light-frame (cold-formed steel) wall systems using flat strap bracing | 14.1 | 4 | 2 | 3½ | NL | NL | 65 | 65 | 65 |
| 20. Cross-laminated timber shear walls | 14.5 | 3 | 3 | 3 | 65 | 65 | 65 | 65 | 65 |
| 21. Cross-laminated timber shear walls with shear resistance provided by high-aspect-ratio panels only | 14.5 | 4 | 3 | 4 | 65 | 65 | 65 | 65 | 65 |
| **B. BUILDING FRAME SYSTEMS** | | | | | | | | | |
| 1. Steel eccentrically braced frames | 14.1 | 8 | 2 | 4 | NL | NL | 160 | 160 | 100 |
| 2. Steel special concentrically braced frames | 14.1 | 6 | [illegible] | [illegible] | [illegible] | [illegible] | [illegible] | [illegible] | [illegible] |

# Seismic Design Factors – (Force Based Design)

- Seismic spectral acceleration is divided by R.
- F=mass times acceleration
- Dividing the acceleration by R = dividing the design force by R.
- Codes allow use of this post yield capacity (R = 6 and 8!!!! fairly common.)
- Way past yield!
- Relies on post yield ductility.

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$$

# Force Based Design

- Well proven method taught in engineering schools.
- Included in most codes and standards (ASCE, AISC, ACI, IBC)
- Uses factors (LRFD, ASD)
- The main idea is to stay under yield or “allowable”.
- Solid methodology but not great at predicting post yield behavior.

# Example new POA Admin Building

- Goes well past yield in MCE
- Has risk of failure in MCE
- Meets code
- Meets professional standard of care
- (Most people don’t understand or care! “Meets code its fine.”)

# Question….Can we utilize ductile capacity for DE?

Answer….Absolutely!

# Study Work

**Studies** Seismic Criteria:

Perform an engineering evaluation of the terminal design using ASCE 61-14 “controlled and repairable damage” seismic performance criteria in design earthquake (DE).Evaluate design details and potential cost savings if “controlled and repairable damage” performance criteria is used in the design earthquake as recommended by the Municipality of Anchorage Geotechnical Advisory Commission (GAC) in 2014. The current performance criteria are inferred to be “minimal damage” at design earthquake due to a 7day to operational / repair timeframe as recommended by the GAC in 2014. Provide basic conceptual details including a cross section of the dock that shows piling, pile caps, deck, and basic connection features. Provide a concept level cost estimate. This shall include a preliminary square foot cost estimate to be used as a comparison to the current project baseline. Evaluate the location and the extent of structural damage that would be expected on the structure. Evaluate the ability of the structure to carry basic service loads under emergency conditions following the expected damage. Provide a narrative describing inspection and repair plans and details following a design event.

Deliverables:

- Terminal design engineering evaluation and preliminary details with “controlled and repairable damage” performance in design event.
- Potential construction cost savings at this performance state.
- Narrative describing expected structural damage with the revised design.
- Narrative describing the ability of the structure to carry service loads under emergency conditions in this performance state.
- Narrative describing of proposed inspection and repair plan for post-design seismic event.

PORT OF ALASKA MODERNIZATION PROGRAM

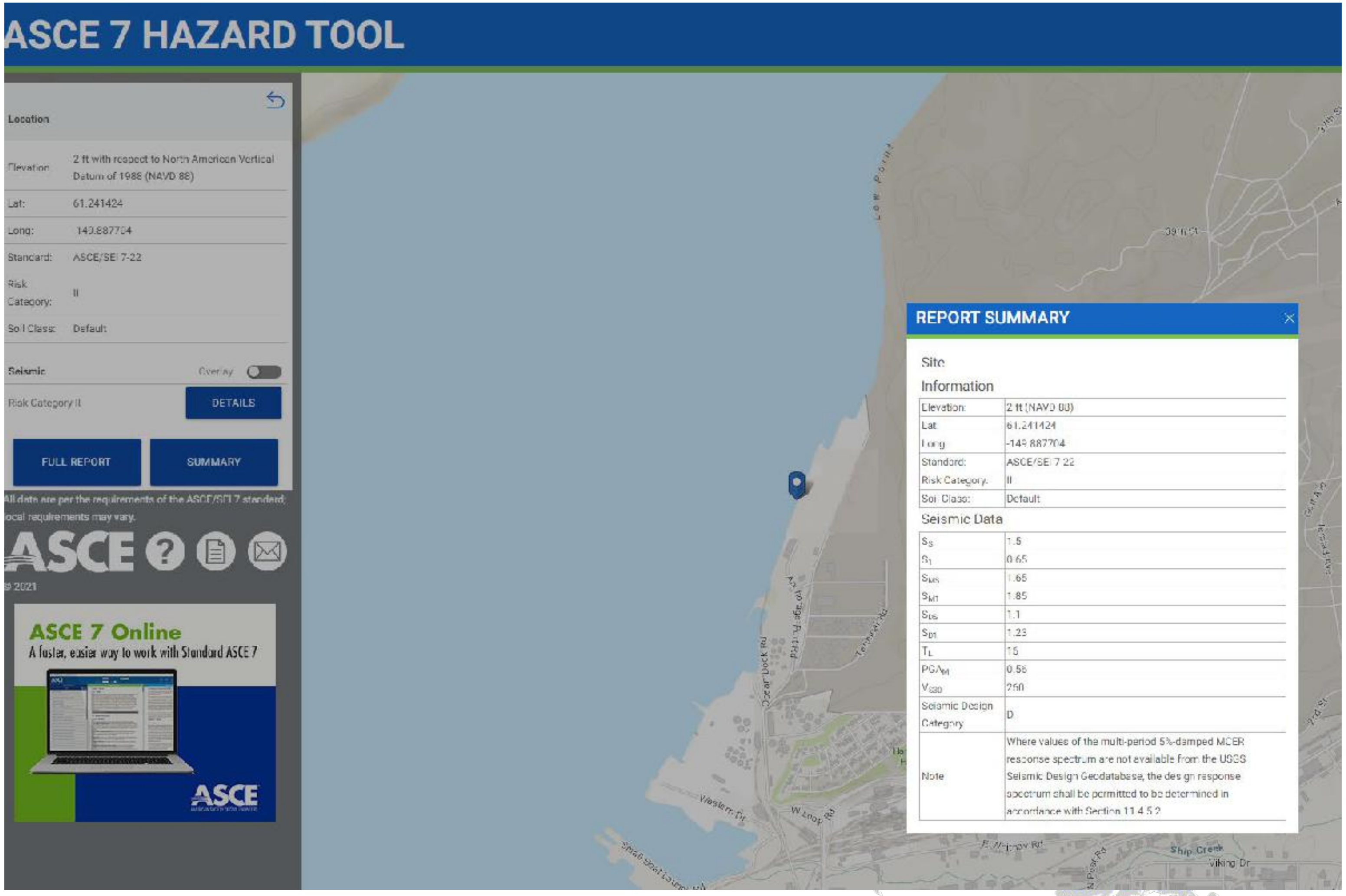
PORT of ALASKA IN ANCHORAGE

# Site Specific Seismic

- Start with USGS mapped values
- Commission study to refine
- Conduct field work
- Update study
- Repeat

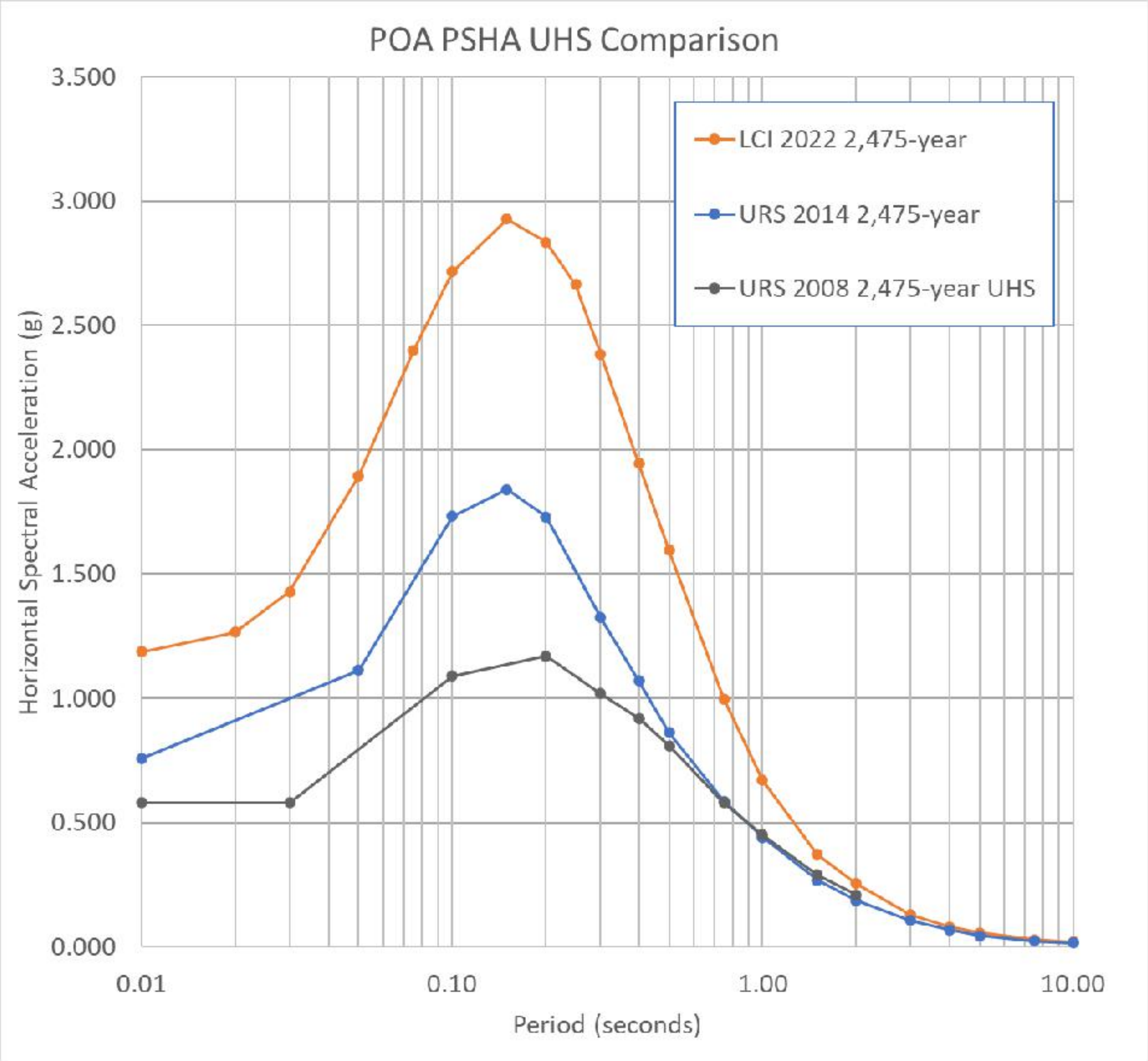
# USGS / ASCE 7 mapped values

- Online tool
- Values being updated



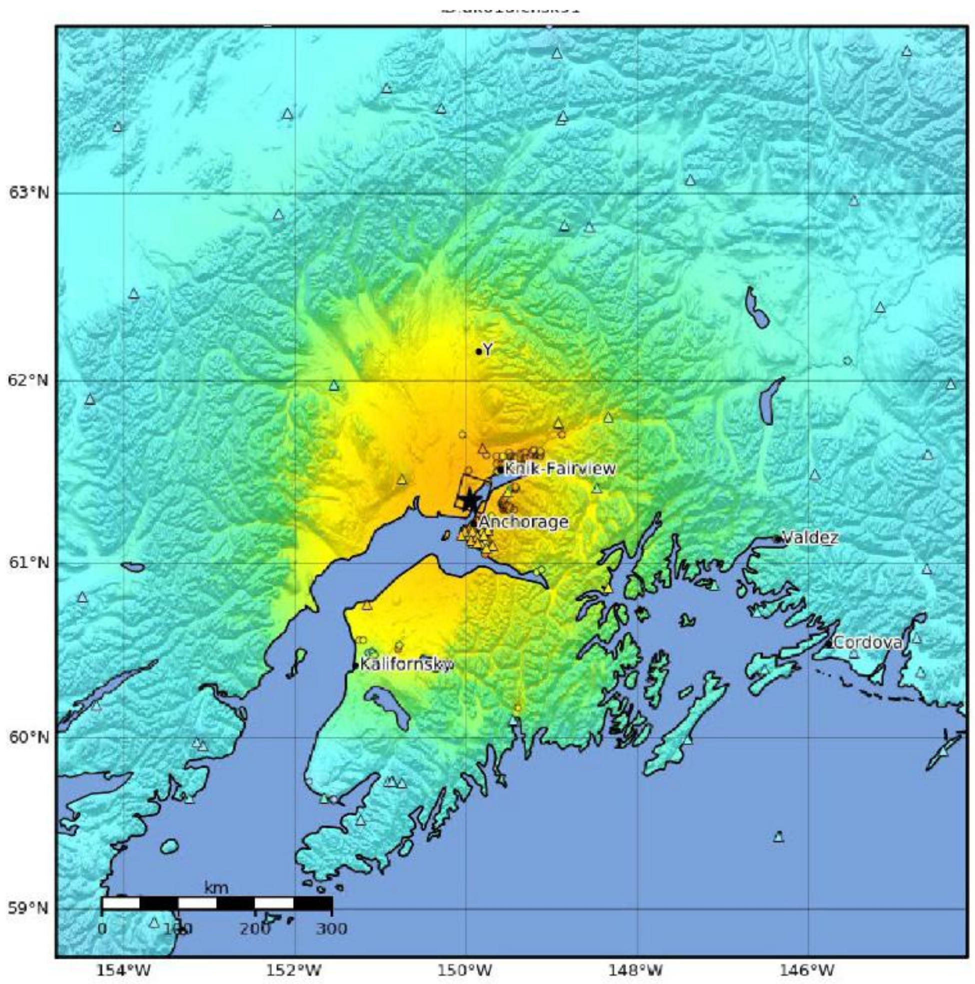
# Previous Hazard Analysis

- 2008 URS
- 2014 URS
- 2022 LCI



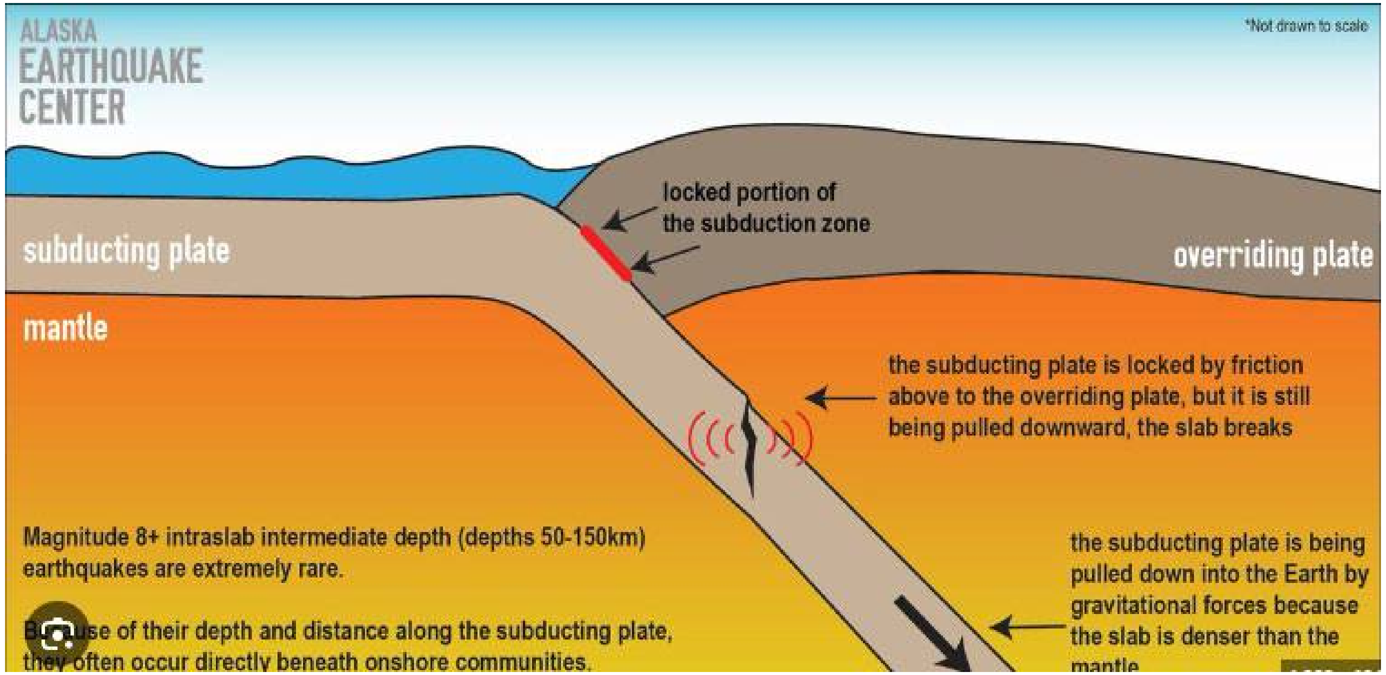
# New Hazard Analysis

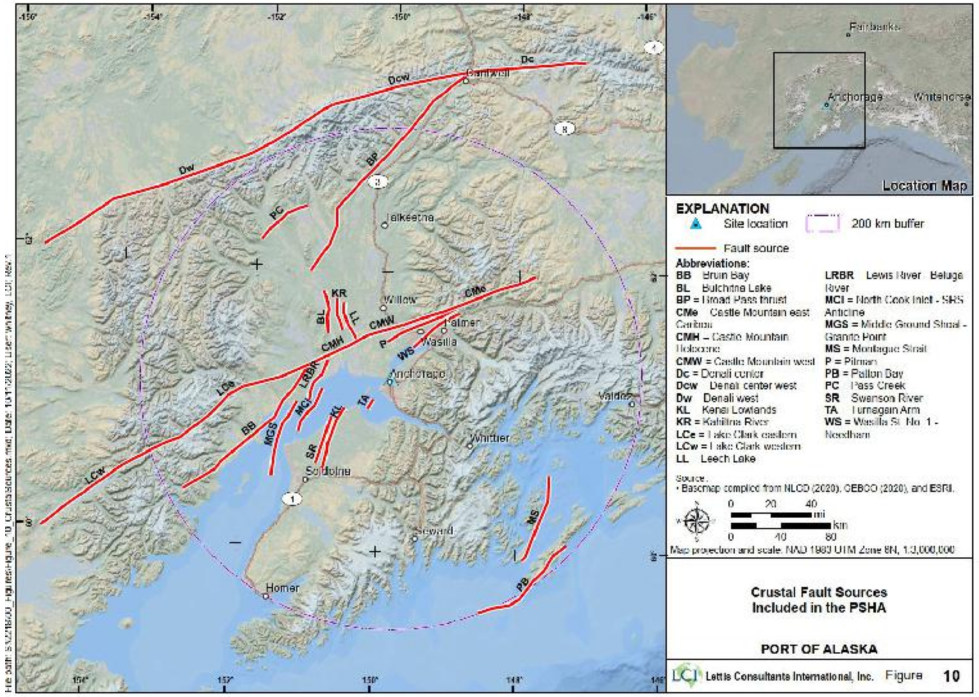
- USGS and others study 2018 Mw 7.1 event
- Lettis Consultants updating values
- USGS updating models



# New Hazard Analysis

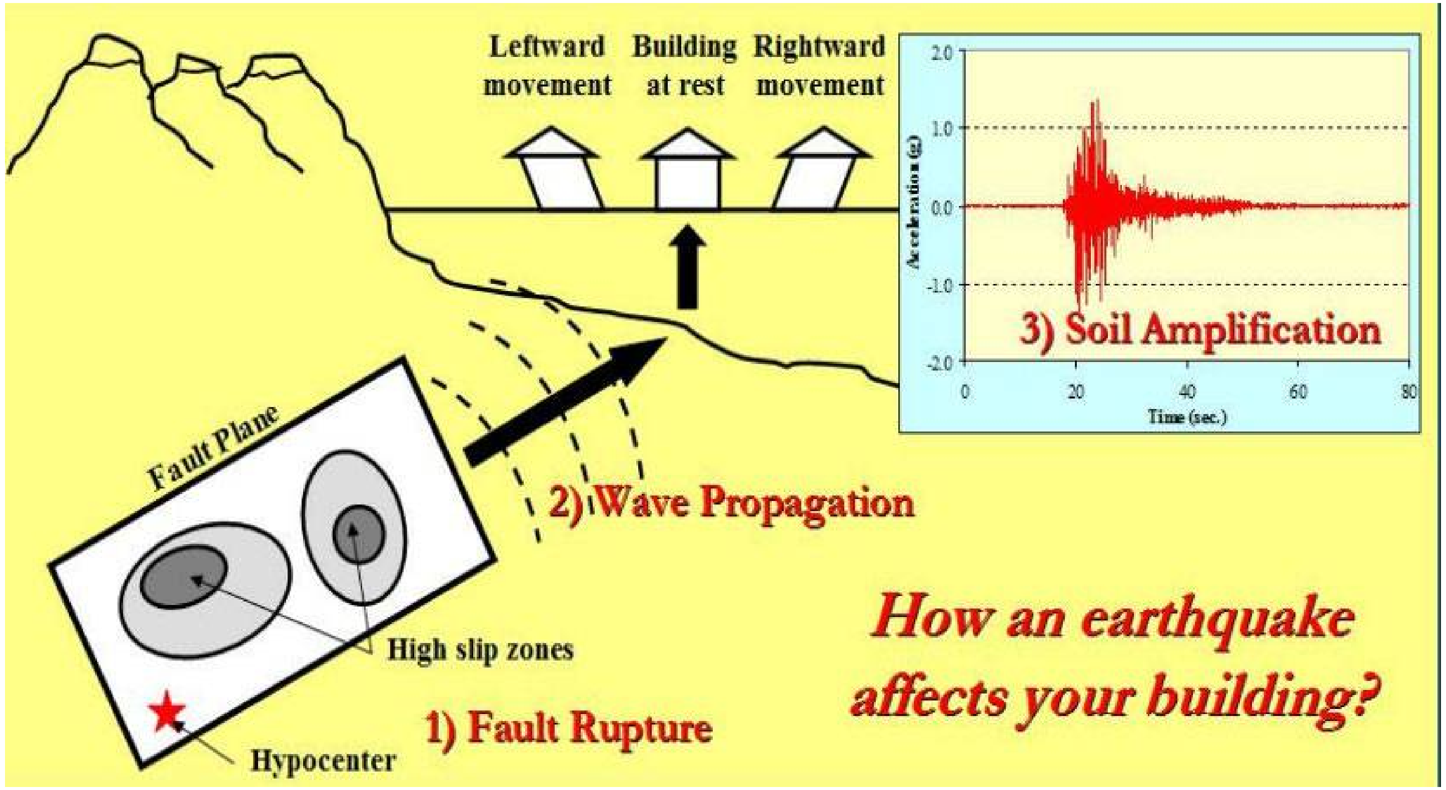
- Seismic sources
- Subduction Zone (Aleutian Mega Thrust)
- Inter-Slab Faults (Castle Mountain)
- Intra-Slab (2018 Event)



# Understanding soil column

- Where is bedrock?
- How many layers are there
- How dense are the layers



# Soil column data gaps

- Glacial till (not rock) is firm ground. This is several hindered feet down
- Previous borings hit gravel layer with artesian water at 150 feet
- Previous brings heaved at this layer and drilling was stopped.
- Current borings using casing advance drill system and hot 265 feet.
- Shear wave velocity from deep borings will define the soil column better

# 2022 drilling program

- Two deep holes.

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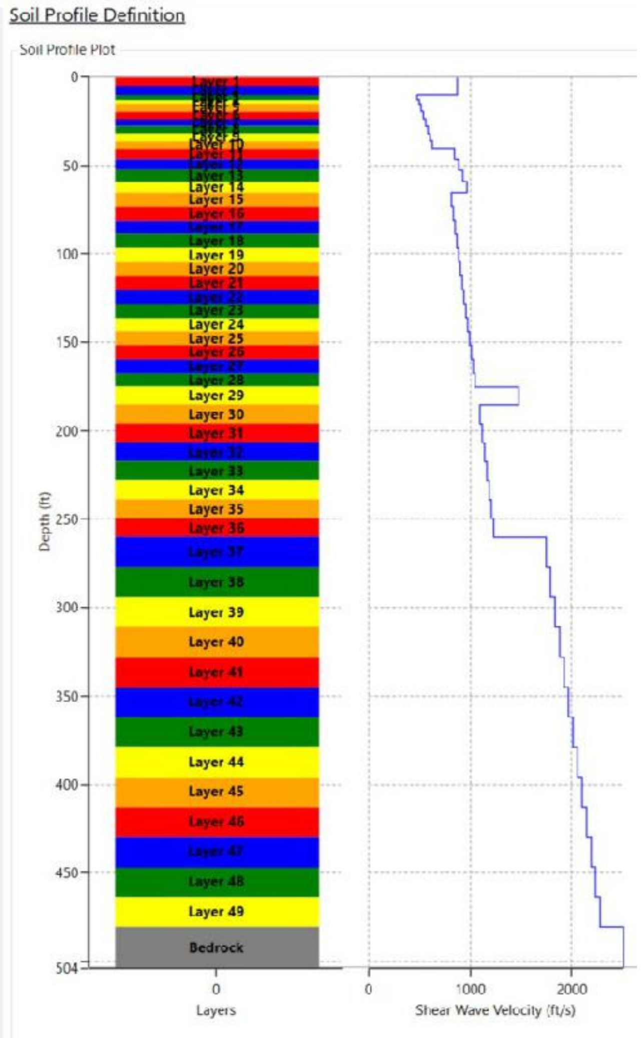
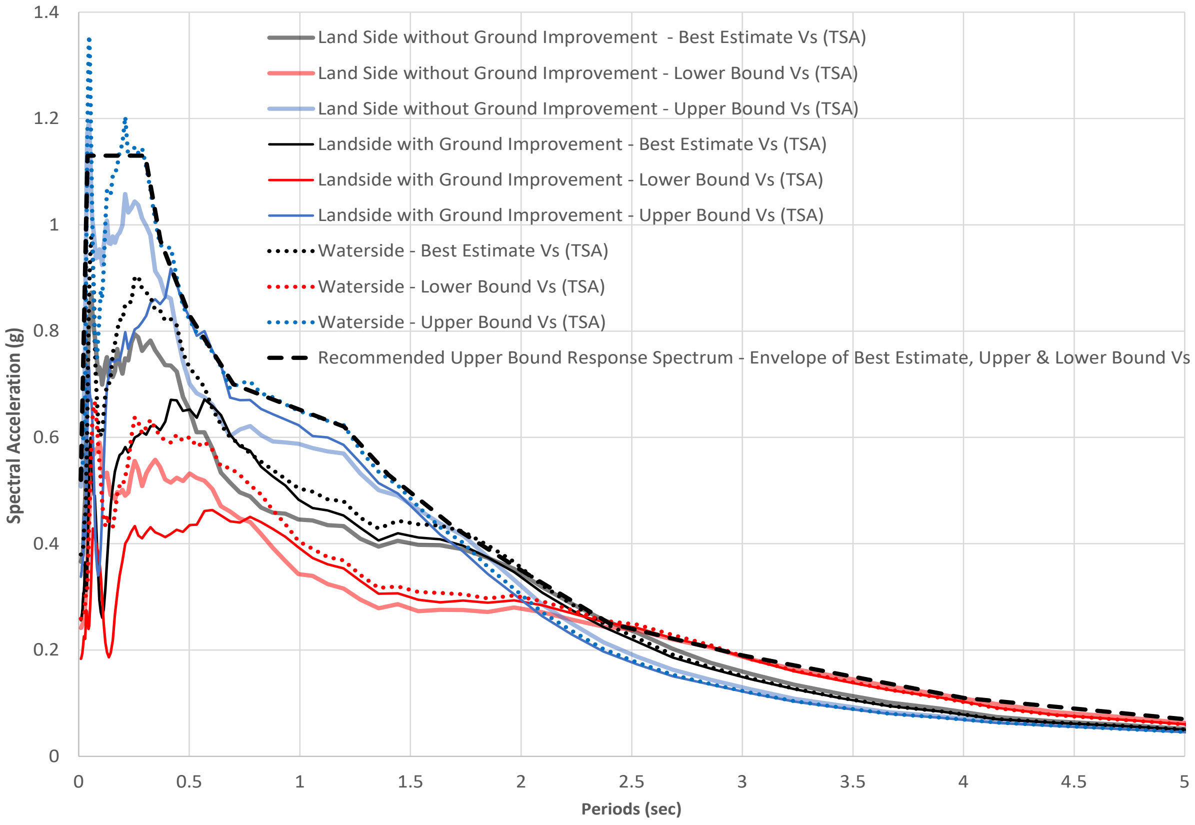
# Updated soil profile and response spectrum



# Science experiment to determine EQ loads

Intra-slab component increased significantly

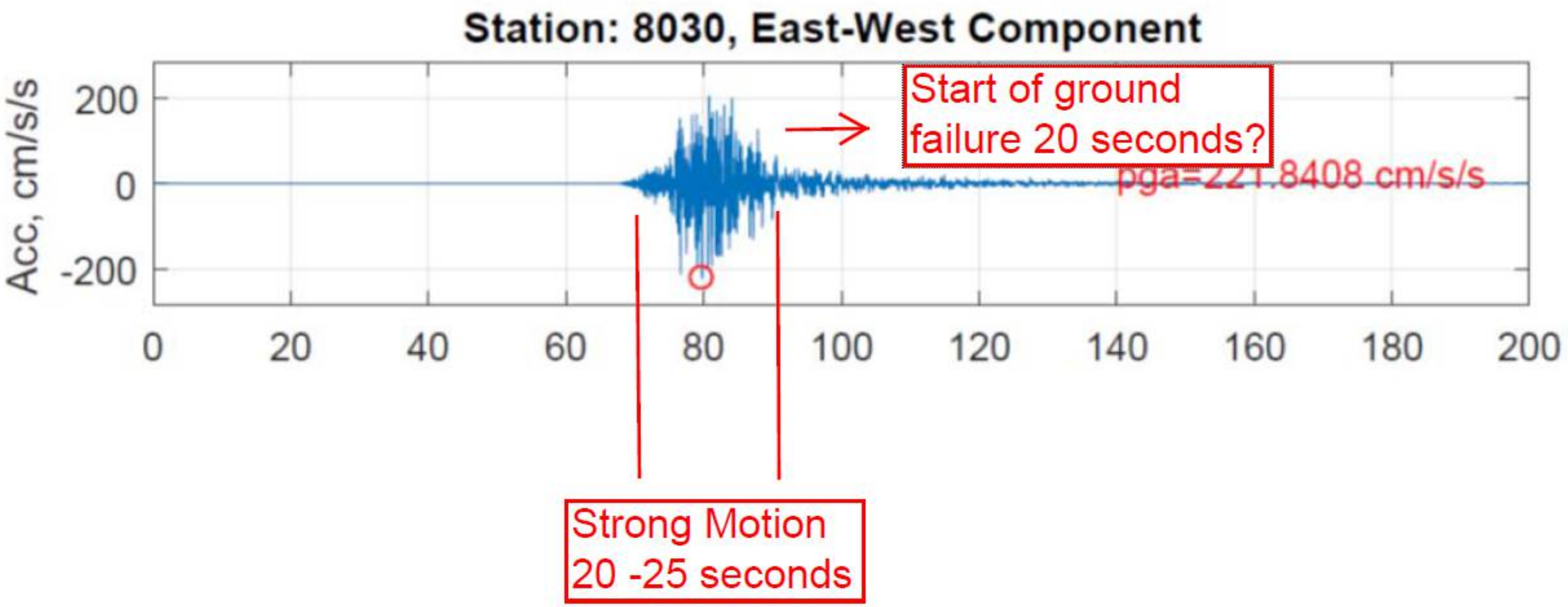
Overall 30% increase over 2014

Upcoming updated USGS may be higher

# Seismic Slope Stability

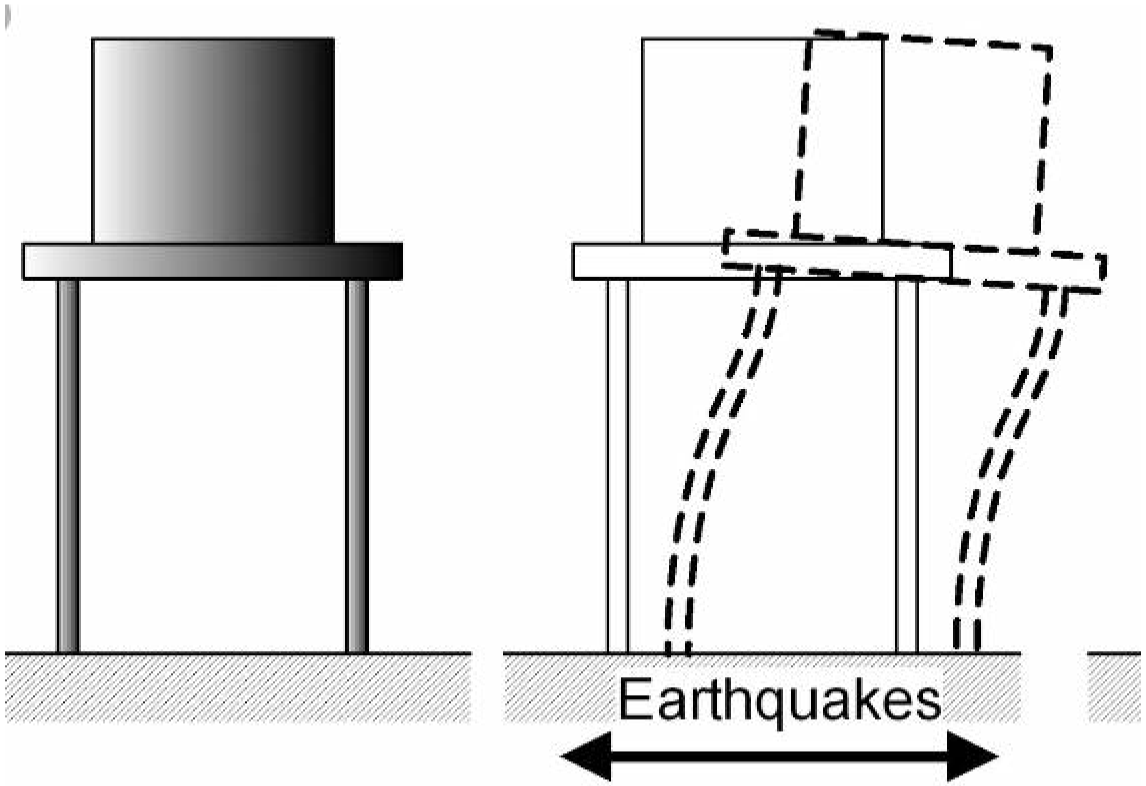
- A risk for waterfront projects

# Combined Inertial and Kinematic November 2018 Anchorage



# Inertial Loads

- Mass of structure responding to ground movement.
- Related to mass and stiffness.
- Cyclical



# Durations

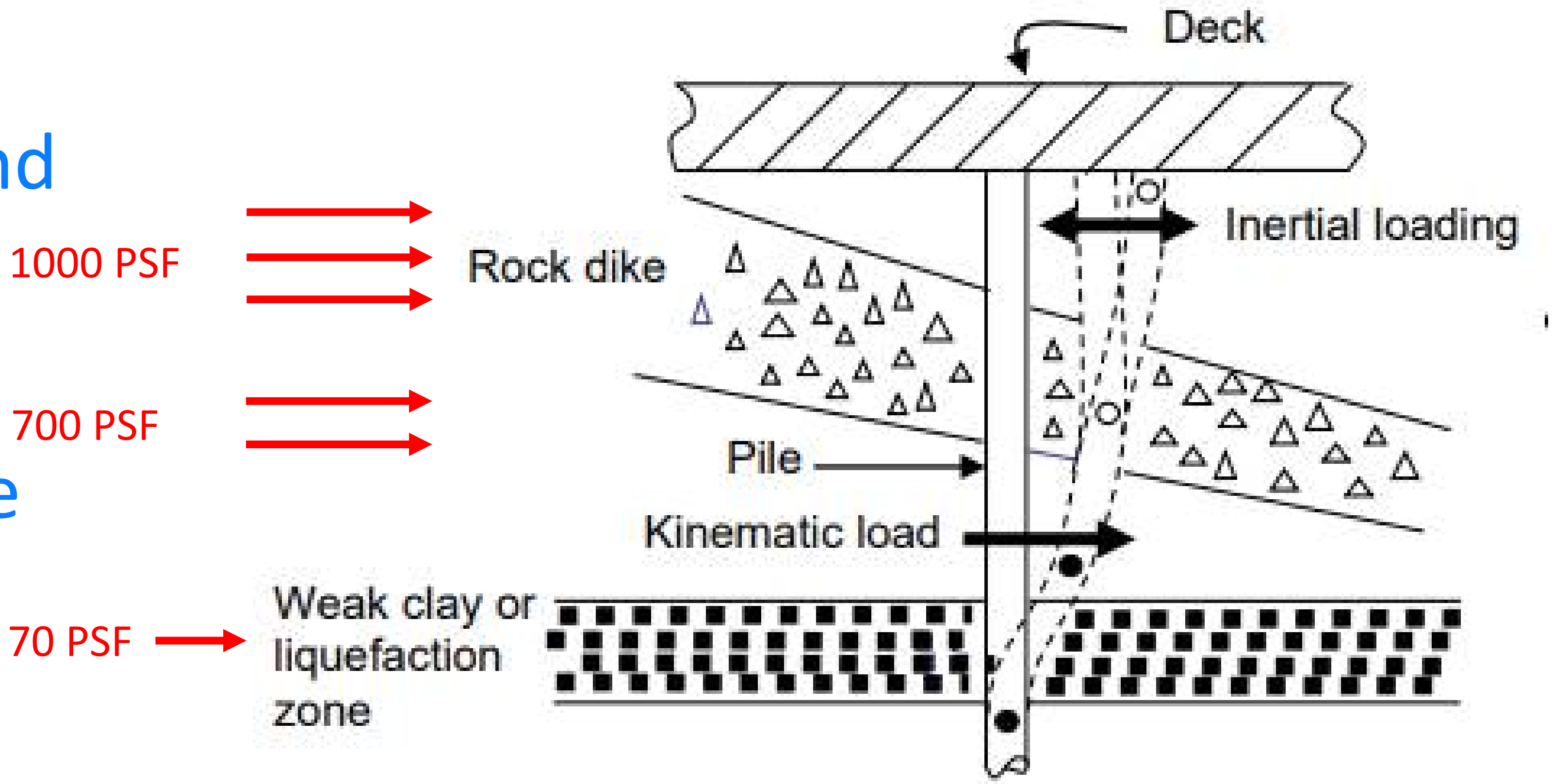
Approximate Peak Ground Acceleration and Duration of Strong-Phase Shaking (California Earthquakes)

| magnitude | maximum acceleration (g) | duration (sec) |
|---|---|---|
| 5.0 | 0.09 | 2 |
| 5.5 | 0.15 | 6 |
| 6.0 | 0.22 | 12 |
| 6.5 | 0.29 | 18 |
| 7.0 | 0.37 | 24 |
| 7.5 | 0.45 | 30 |
| 8.0 | 0.50 | 34 |
| 8.5 | 0.50 | 37 |

Liquefaction threshold?

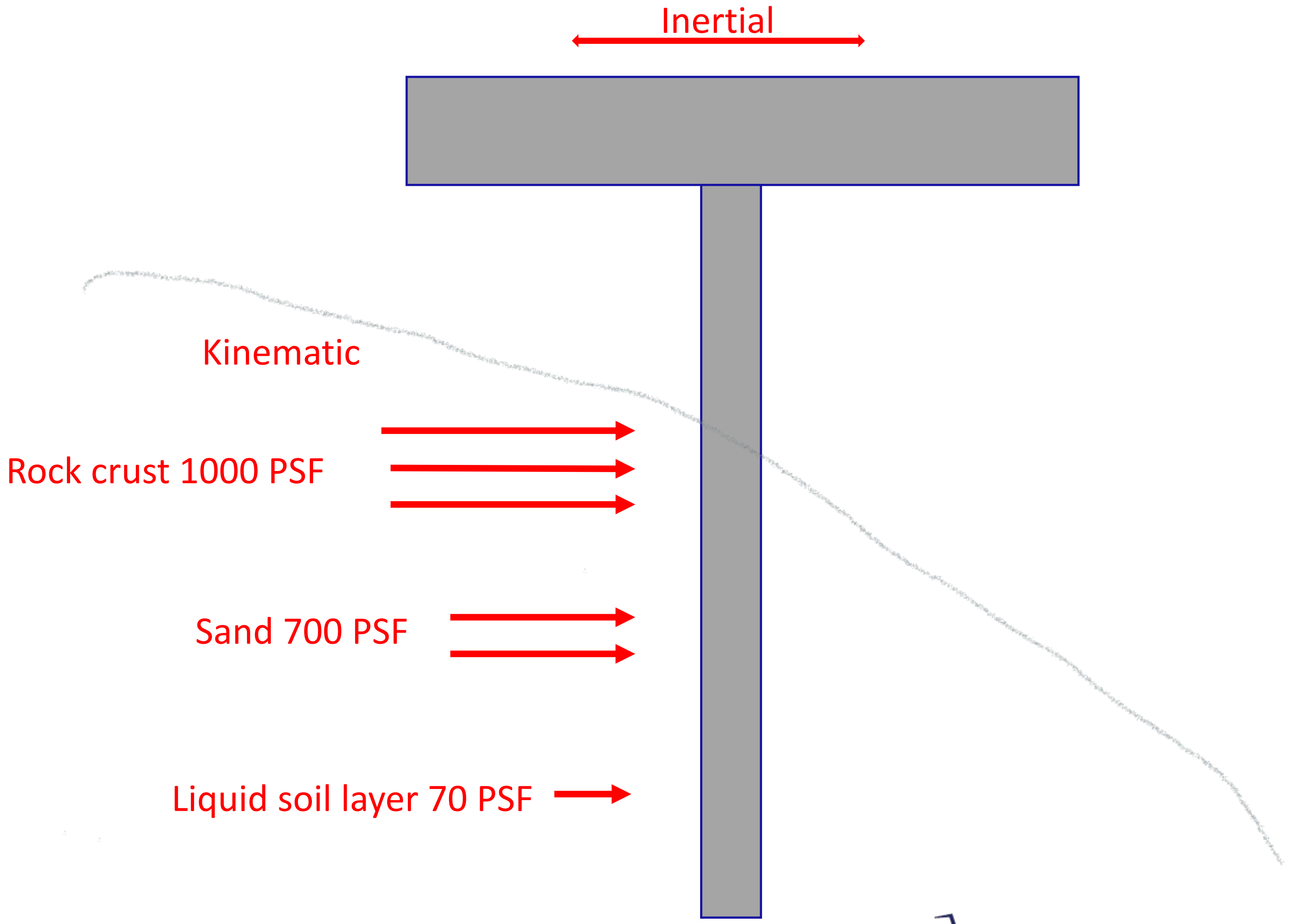
# Kinematic Loads

- Monatomic load
- Different  type and location from seismic load
- Separated in time for most events



# Kinematic Loads

- Moving soil pushing on piling



# Kinematic Loads

- Moving soil
- (2010 Chile event)

# Combined Inertial and Kinematic

- Short duration Earthquake - ground failure occurs after most of strong motion is over.
- Long duration Earthquake - combines strong motion and ground failure at the same time!

Retaining Wall Failure
Kings Harbor Marina, Redondo Beach
1994 Northridge, M 6.7

# February 2010 Maule, Chile Earthquake
# Magnitude 8.8 Ground Failure/Lateral Spreading Port of Coronel

PORT OF ALASKA MODERNIZATION PROGRAM

PORT of ALASKA IN ANCHORAGE

1995 Kobe Japan Mw 6.9

Many large container cranes were damaged on Rokko Island. The damage to the cranes is primarily due to rails spreading and settling. Crane damage consisted of buckling of legs at the portal ties.

1995 Kobe Japan Mw 6.9
Liquefaction and lateral spreading damaged the crane rails

# Lateral Spreading – Bulkhead Failure
# 1995 Magnitude 6.9 Kobe Japan

# Lateral Spreading @ Port of Alaska 2018


PORT OF ALASKA MODERNIZATION PROGRAM

PORT of ALASKA IN ANCHORAGE

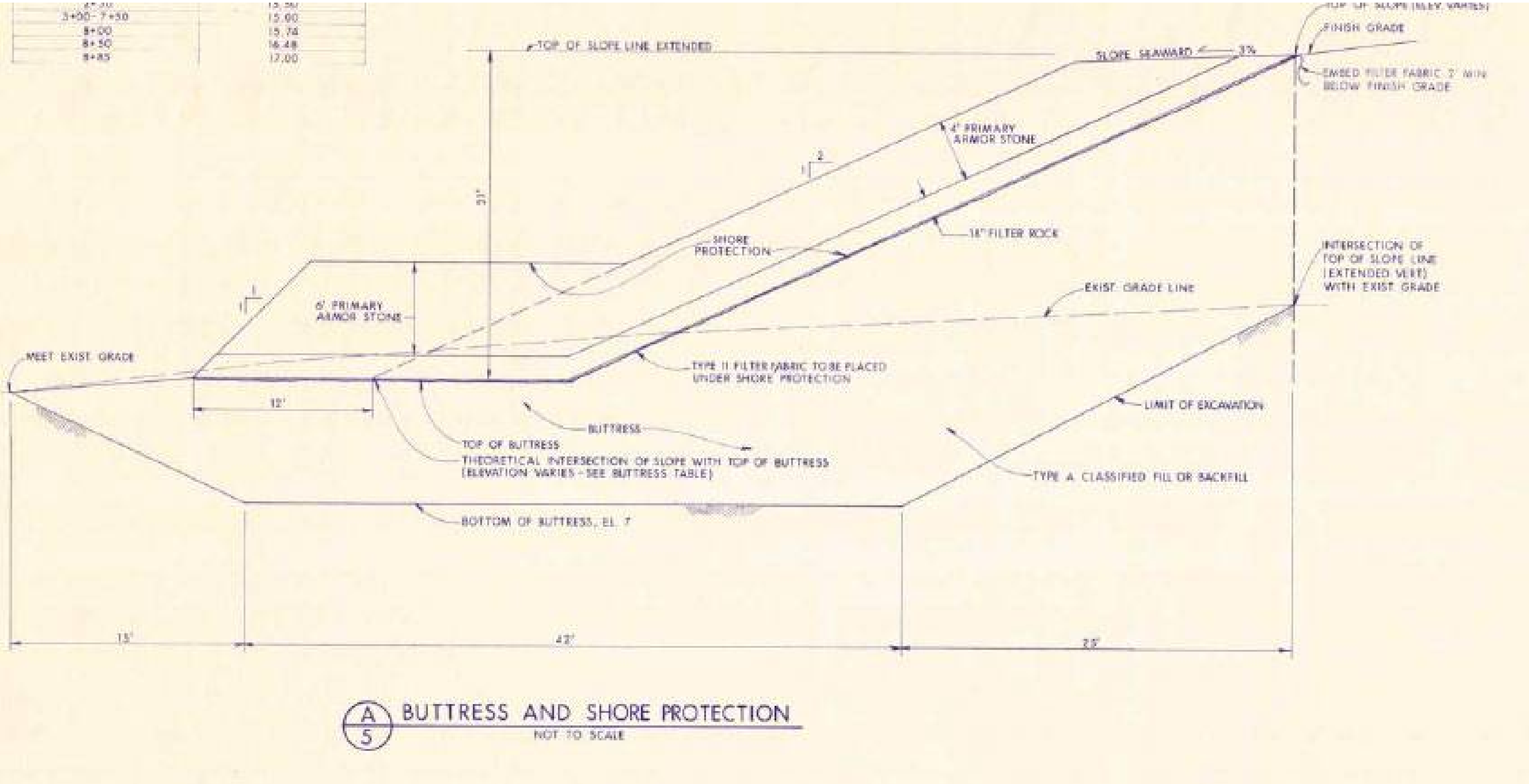
# Sand Boils Port of Alaska
# 2018 Anchorage, M 7.1

# Sand Boils Port of Alaska
# 2018 Anchorage, M 7.1

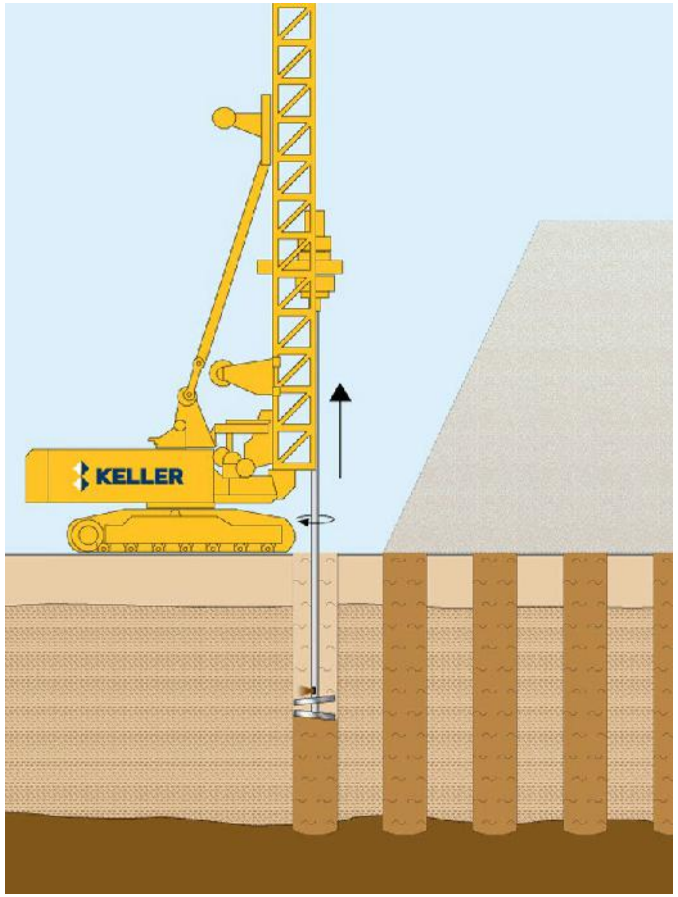
# How to resist these types of forces?

- Engineered Slopes
- Ground Improvements
- Bulkheads

# Engineered Slopes 1990s POA Transit Yard



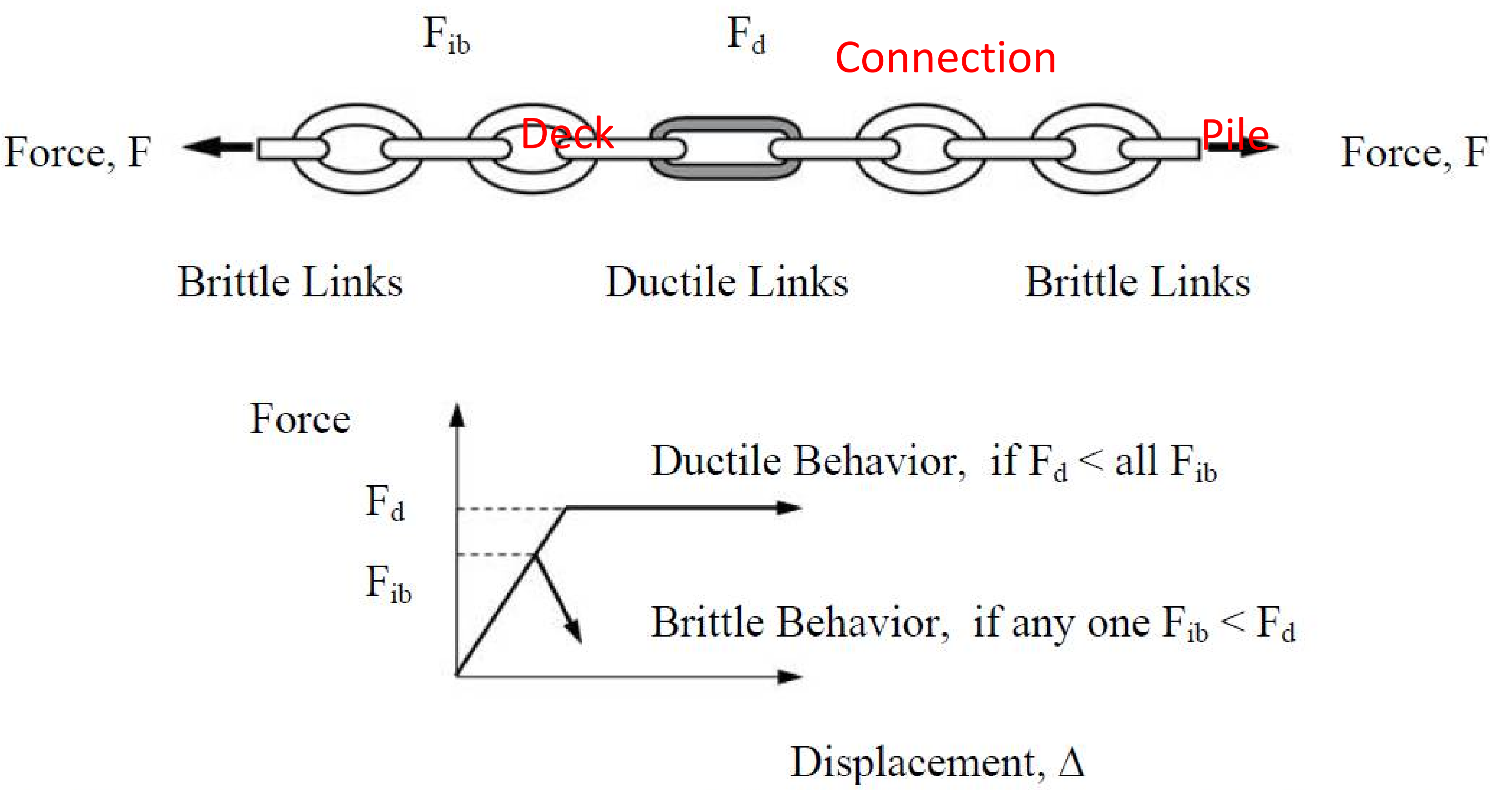
# Engineered Slopes - Deep Soil Mixing




# Structural Ductile Detailing

- Required to achieve desired performance.

# Ductile Fuse Concept



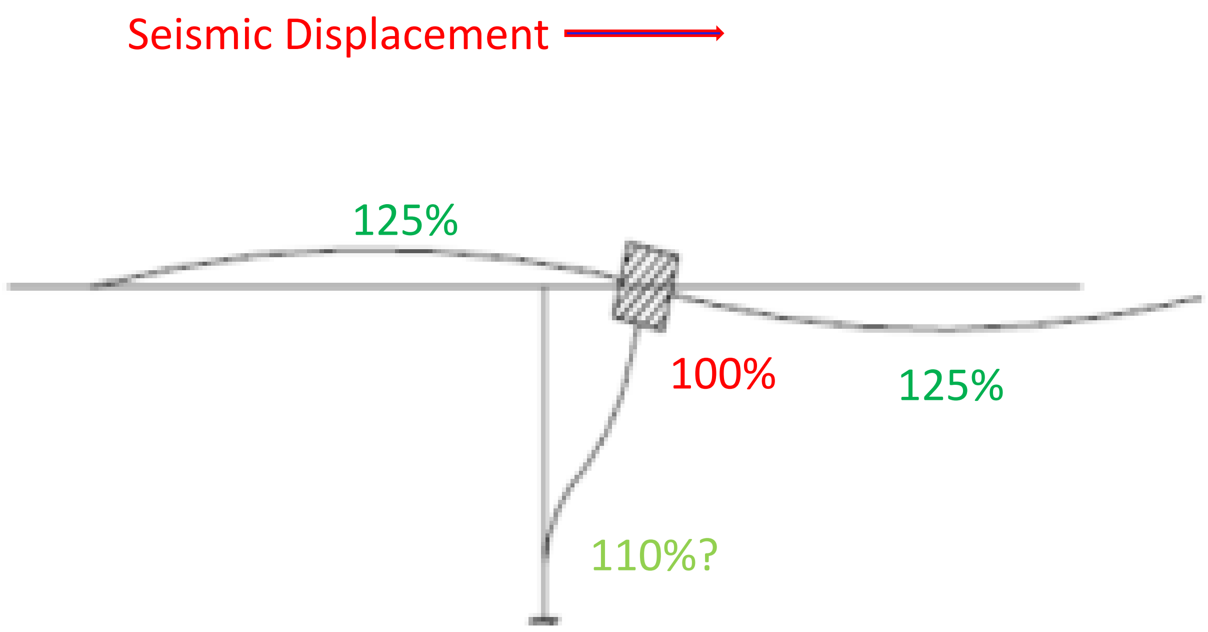

Chain Analogy for Capacity-Protected Design (after Paulay and Priestley, 1992)

# Ductile Fuse Concept

- Must identify the yielding element
- Must protect non-yielding elements

# Displacement Based Design

- Use expected materials properties
- Impart a displacement in model
- Yielding element will "jump out"
- Deck needs more capacity than hinge.



# Expected Materials Properties

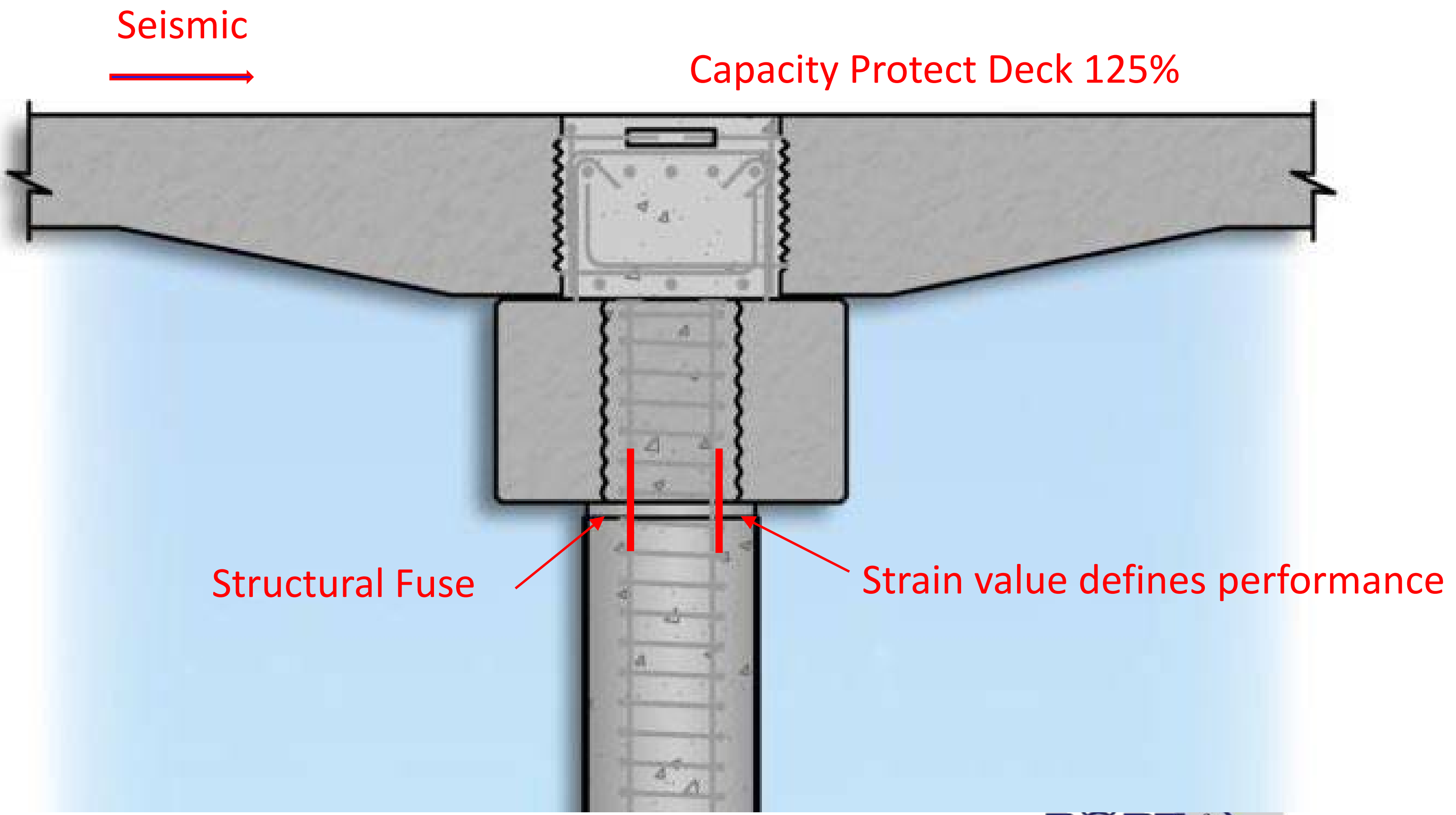
- AISC 341-16
- Yield and tensile strength greater than design values

**TABLE A3.1**
**$R_y$ and $R_t$ Values for Steel and Steel Reinforcement Materials**

| Application | $R_y$ | $R_t$ |
|---|---|---|
| Hot-rolled structural shapes and bars: | | |
| • ASTM A36/A36M | 1.5 | 1.2 |
| • ASTM A1043/A1043M Gr. 36 (250) | 1.3 | 1.1 |
| • ASTM A992/A992M | 1.1 | 1.1 |
| • ASTM A572/A572M Gr. 50 (345) or 55 (380) | 1.1 | 1.1 |
| • ASTM A913/A913M Gr. 50 (345), 60 (415), 65 (450), or 70 (485) | 1.1 | 1.1 |
| • ASTM A588/A588M | 1.1 | 1.1 |
| • ASTM A1043/A1043M Gr. 50 (345) | 1.2 | 1.1 |
| • ASTM A529 Gr. 50 (345) | 1.2 | 1.2 |
| • ASTM A529 Gr. 55 (380) | 1.1 | 1.2 |
| Hollow structural sections (HSS): | | |
| • ASTM A500/A500M Gr. B | 1.4 | 1.3 |
| • ASTM A500/A500M Gr. C | 1.3 | 1.2 |
| • ASTM A501/A501M | 1.4 | 1.3 |
| • ASTM A53/A53M | 1.6 | 1.2 |
| • ASTM A1085/A1085M | 1.25 | 1.15 |
| Plates, Strips and Sheets: | | |
| • ASTM A36/A36M | 1.3 | 1.2 |
| • ASTM A1043/A1043M Gr. 36 (250) | 1.3 | 1.1 |
| • ASTM A1011/A1011M HSLAS Gr. 55 (380) | 1.1 | 1.1 |
| • ASTM A572/A572M Gr. 42 (290) | 1.3 | 1.0 |
| • ASTM A572/A572M Gr. 50 (345), Gr. 55 (380) | 1.1 | 1.2 |
| • ASTM A588/A588M | 1.1 | 1.2 |
| • ASTM A1043/A1043M Gr. 50 (345) | 1.2 | 1.1 |
| Steel Reinforcement: | | |
| • ASTM A615/A615M Gr. 60 (420) | 1.2 | 1.2 |
| • ASTM A615/A615M Gr. 75 (520) and Gr. 80 (550) | 1.1 | 1.2 |
| • ASTM A706/A706M Gr. 60 (420) and Gr. 80 (550) | 1.2 | 1.2 |

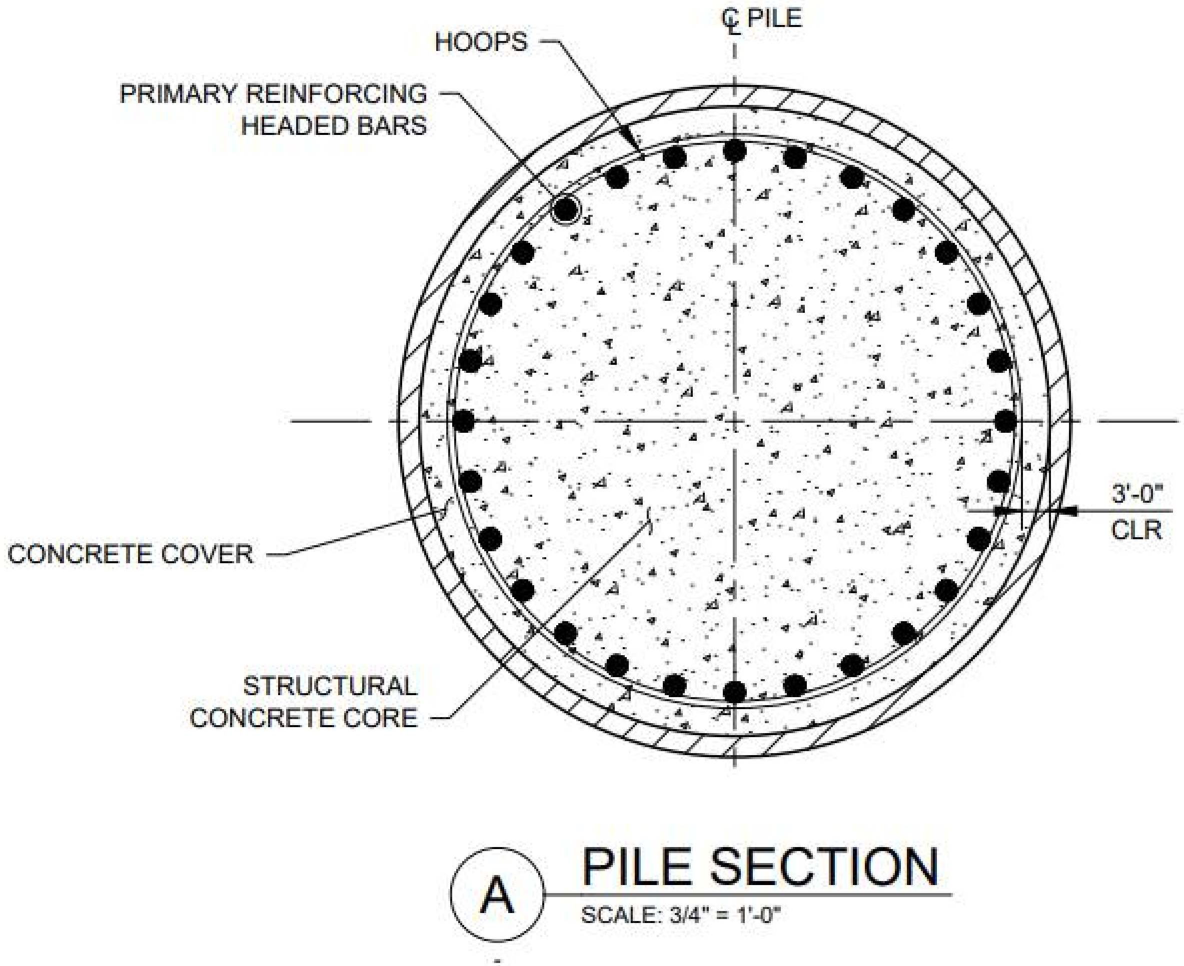
# POLA Code

- Strong Deck - Weak Pile ductile moment frame.
- Structural fuse at pile to deck connection.
- Deck is capacity protected.



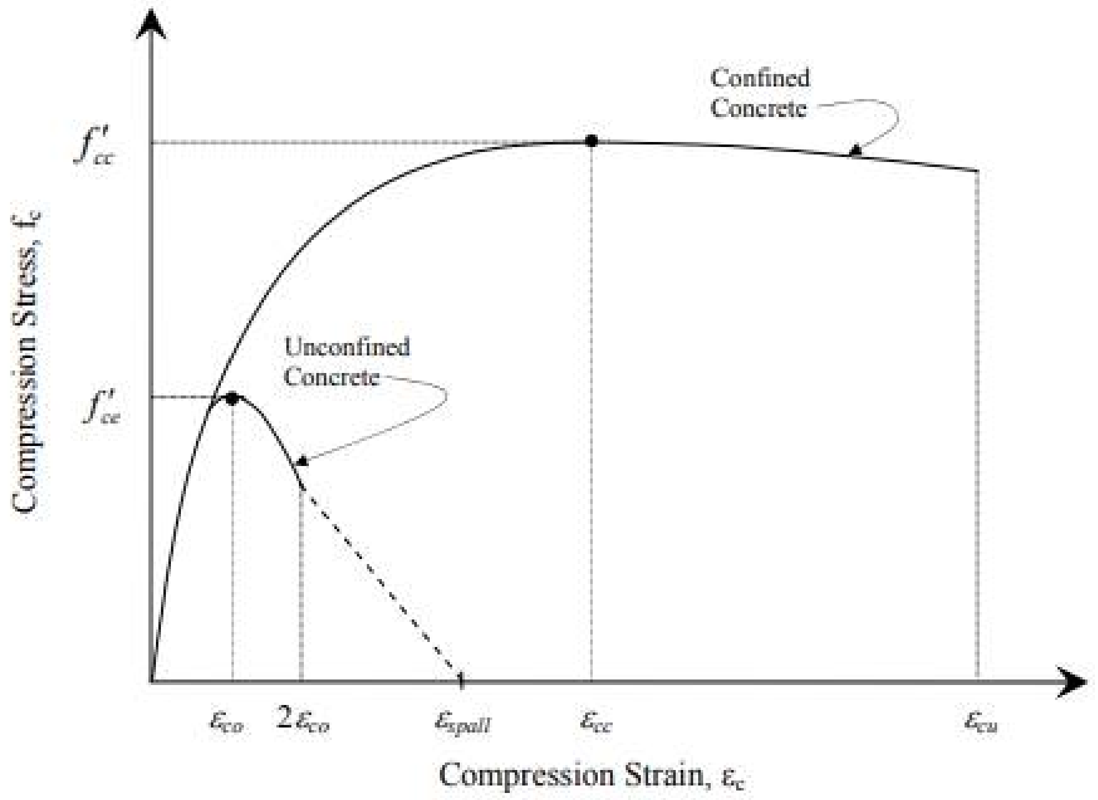
# Composite Pile

- Need to understand post yield behavior of pile to deck connection
- Composite section with several materials
- Push each material past yield
- Nonlinear and complicated



# Confined Concrete

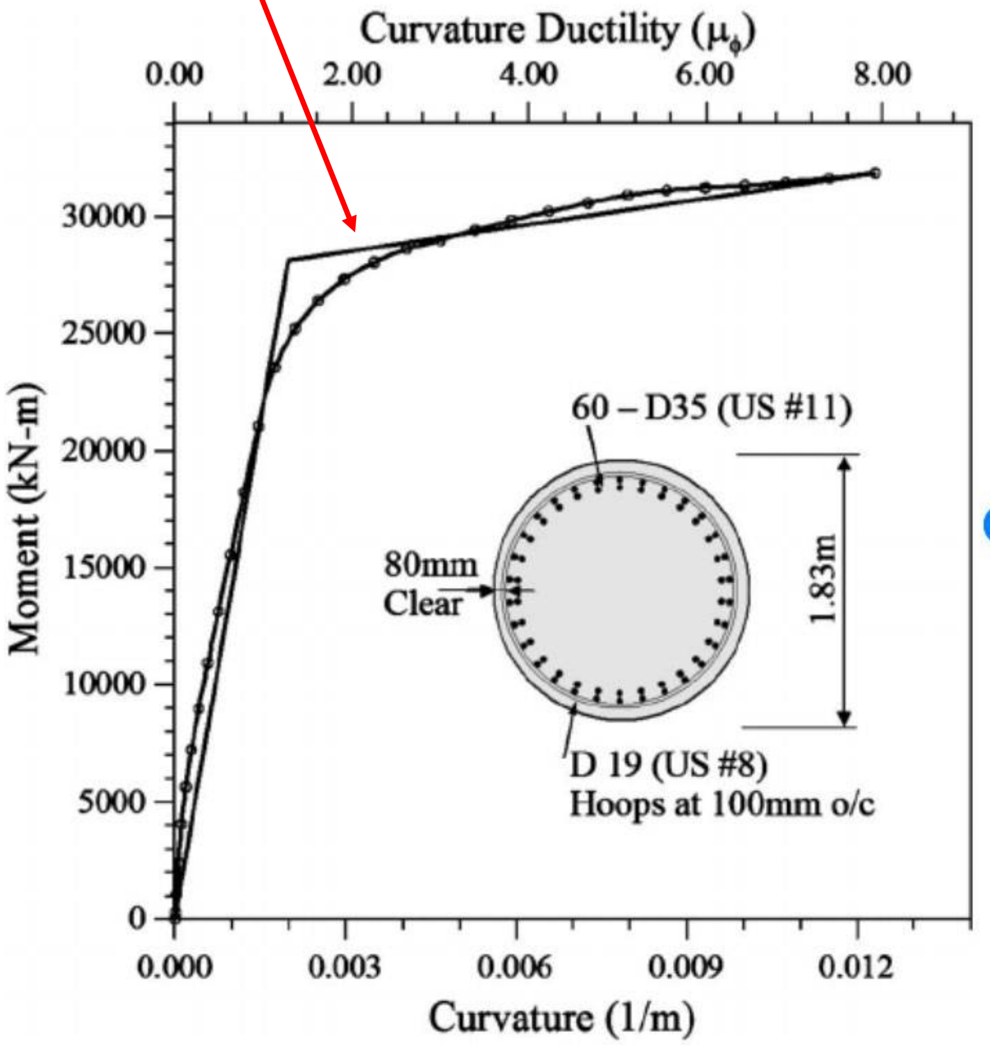
- Mander and Park model for confined and unconfined concrete
- Confined concrete can be ductile!



# Computer Analysis

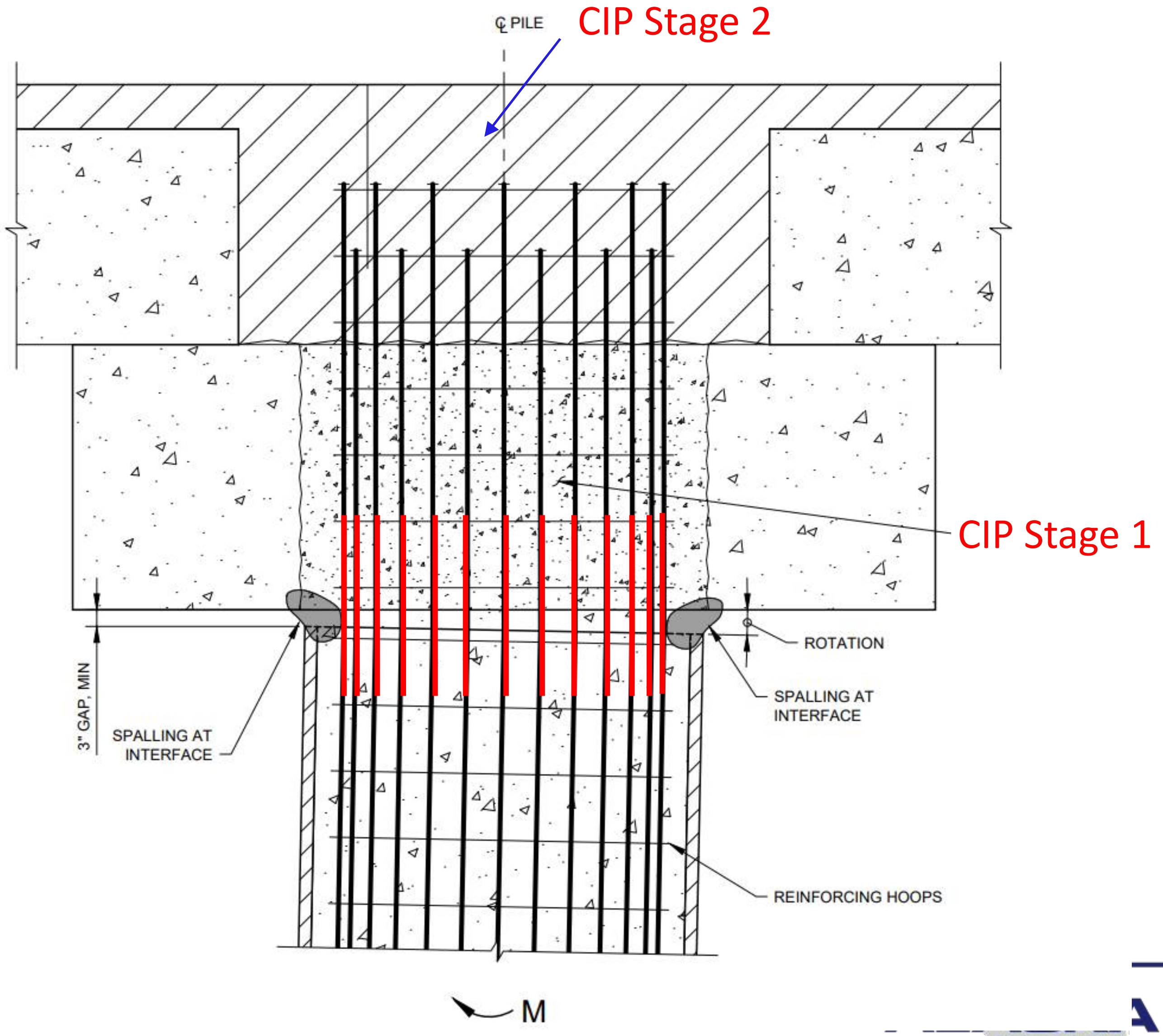
- Need moment curvature properties of composite section ductile hinge
- Use computer program such as Xtract
- (Similar to stress strain curve but different.)

Bi-Linear Curve



# Engineered Hinge

- Deck capacity protected
- Spalling at pile to cap interface, primarily in cover
- Limited strain in primary reinforcing
- Concrete core remains essentially intact
- No buckling of primary reinforcing



# Strain Limits (ASCE 61-14 chapter 3)

## Minimal Damage (near elastic)

Table 3-1. Strain Limits for "Minimal Damage" per Section 2.4.3

| | | Hinge location | | |
|---|---|---|---|---|
| Pile type | Component | Top of pile | In ground | Deep in ground (>10$D_p$) |
| Solid concrete pile | Concrete | $\varepsilon_c \le 0.005$ | $\varepsilon_c \le 0.005$ | $\varepsilon_c \le 0.008$ |
| | Reinforcing steel | $\varepsilon_s \le 0.015$ | | |
| | Prestressing steel | | $\varepsilon_p \le 0.015$ | $\varepsilon_p \le 0.015$ |
| Hollow concrete pile[a] | Concrete | $\varepsilon_c \le 0.004$ | $\varepsilon_c \le 0.004$ | $\varepsilon_c \le 0.008$ |
| | Reinforcing steel | $\varepsilon_s \le 0.015$ | | |
| | Prestressing steel | | $\varepsilon_p \le 0.015$ | $\varepsilon_p \le 0.015$ |
| Steel pipe pile | Steel pipe | | $\varepsilon_s \le 0.010$ | $\varepsilon_s \le 0.010$ |
| | Concrete | $\varepsilon_c \le 0.010$ | | |
| | Reinforcing steel | $\varepsilon_s \le 0.015$ | | |

[a] If the interior of the hollow pile is filled with concrete, all strain limits shall be the same as for solid piles.

## Controlled and Repairable Damage

Table 3-2. Strain Limits for "Controlled and Repairable Damage" per Section 2.4.2

| | | Hinge location | | |
|---|---|---|---|---|
| Pile type | Component | Top of pile | In ground | Deep in ground (>10$D_p$) |
| Solid concrete pile | Concrete | $\varepsilon_c \le 0.005 + 1.1\rho_s \le 0.025$ | $\varepsilon_c \le 0.005 + 1.1\rho_s \le 0.008$ | $\varepsilon_c \le 0.012$ |
| | Reinforcing steel | $\varepsilon_s \le 0.6\varepsilon_{smd} \le 0.06$ | | |
| | Prestressing steel | | $\varepsilon_p \le 0.025$ | $\varepsilon_p \le 0.025$ |
| Hollow concrete pile[a] | Concrete | $\varepsilon_c \le 0.006$ | $\varepsilon_c \le 0.006$ | $\varepsilon_c \le 0.006$ |
| | Reinforcing steel | $\varepsilon_s \le 0.6\varepsilon_{smd} \le 0.04$ | | |
| | Prestressing steel | | $\varepsilon_p \le 0.020$ | $\varepsilon_p \le 0.025$ |
| Steel pipe pile | Steel pipe | | $\varepsilon_s \le 0.025$[b] | $\varepsilon_s \le 0.035$ |
| | Concrete | $\varepsilon_c \le 0.025$ | | |
| | Reinforcing steel | $\varepsilon_s \le 0.6\varepsilon_{smd} \le 0.06$ | | |

[a] If the interior of the hollow pile is filled with concrete, all strain limits shall be the same as for solid piles.
[b] If the steel pipe pile is infilled with concrete, a value of 0.035 may be used.

## Life Safety Protection

Table 3-3. Strain Limits for "Life Safety Protection" per Section 2.4.1

| | | Hinge location | | |
|---|---|---|---|---|
| Pile type | Component | Top of pile | In ground | Deep in ground (>10$D_p$) |
| Solid concrete pile | Concrete | No limit | $\varepsilon_c \le 0.005 + 1.1\rho_s \le 0.012$ | No limit |
| | Reinforcing steel | $\varepsilon_s \le 0.8\varepsilon_{smd} \le 0.08$ | | |
| | Prestressing steel | | $\varepsilon_p \le 0.035$ | $\varepsilon_p \le 0.050$ |
| Hollow concrete pile[a] | Concrete | $\varepsilon_c \le 0.008$ | $\varepsilon_c \le 0.008$ | $\varepsilon_c \le 0.008$ |
| | Reinforcing steel | $\varepsilon_s \le 0.8\varepsilon_{smd} \le 0.06$ | | |
| | Prestressing steel | | $\varepsilon_p \le 0.025$ | $\varepsilon_p \le 0.050$ |
| Steel pipe pile | Steel pipe | | $\varepsilon_s \le 0.035$[b] | $\varepsilon_s \le 0.050$ |
| | Concrete | No limit | | |
| | Reinforcing steel | $\varepsilon_s \le 0.8\varepsilon_{smd} \le 0.08$ | | |

[a] If the interior of the hollow pile is filled with concrete, all strain limits shall be the same as for solid piles.
[b] If the steel pipe pile is infilled with concrete, a value of 0.050 may be used.

# Strain Limits and Performance

- How much damage would be “repairable”?
- How would it be repaired?
- How long would repair take?
- Engineering design parameter versus maintenance and operational parameter

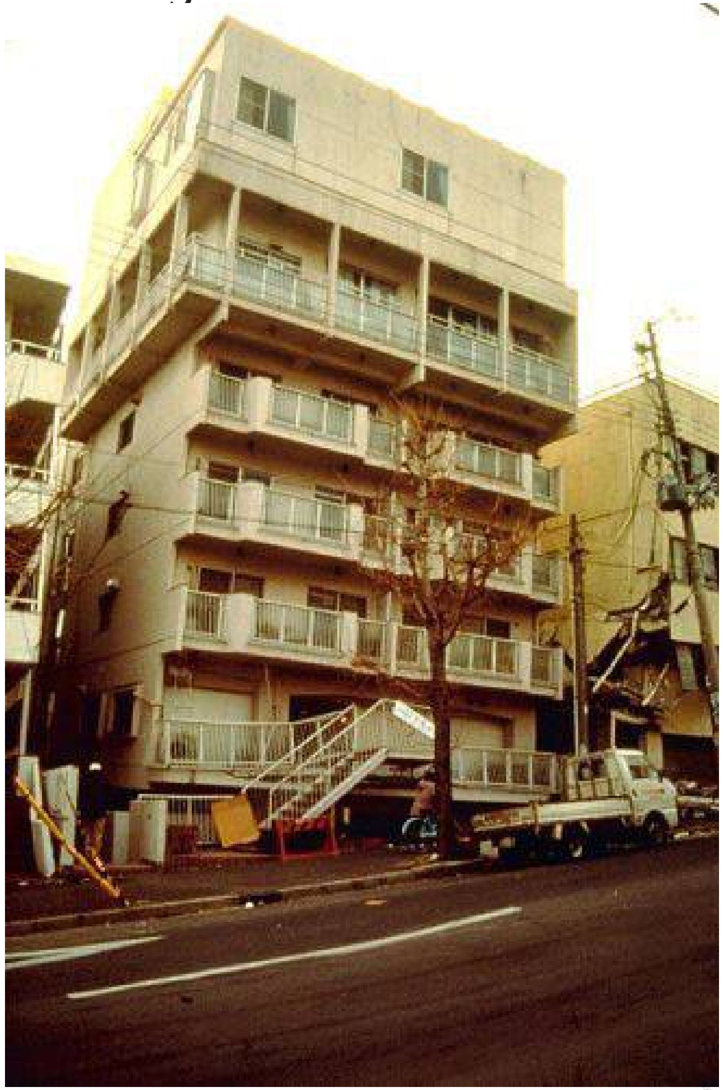
# Ductile Concrete (Northridge 1994 Mw 6.7)

Before


After

1995 Kobe Japan Mw 6.9

Five-year-old 6-story concrete frame with garage level collapse. This was an exception to the rule of good performance of newer concrete buildings.



1995 Kobe Japan Mw 6.9
Five-year-old 6-story concrete frame with garage level collapse. Ductile detailing problems in the columns are shown.

1995 Kobe Japan Mw 6.9

Perhaps the most memorable image flashed around the world after the earthquake, was a bridge on the Hanshin expressway which "rolled over." This is an aerial view of that collapsed section of the Hanshin expressway. This spectacular failure occurred at the location where the superstructure deck changed from steel to concrete.

1995 Kobe Japan Mw 6.9

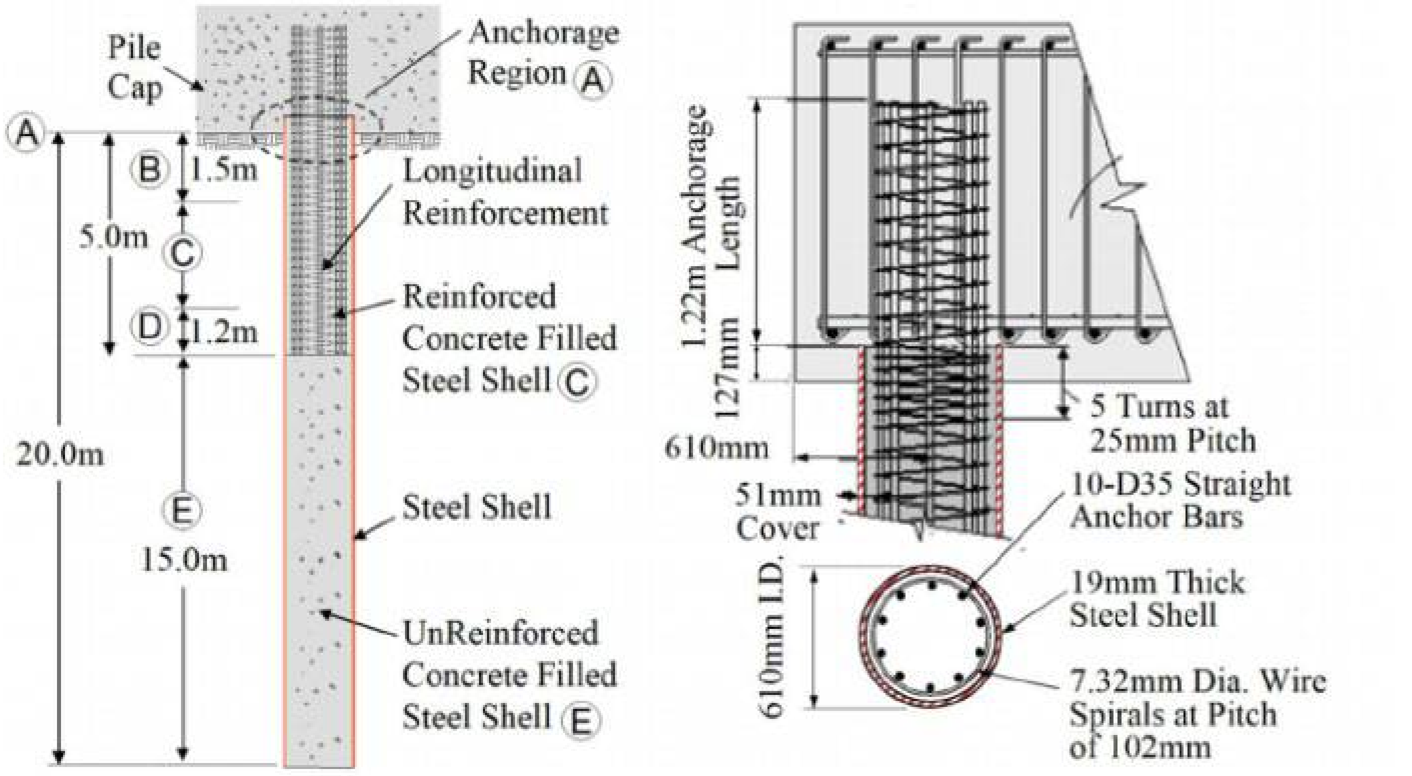
The columns in this segment of the Hanshin expressway are cast monolithically. Between each of these segments there is a simple span deck section which is connected by four bolts across the joint. The whole deck remained intact; none of the segments pulled apart.

1995 Kobe Japan Mw 6.9

Nearly every column along the elevated Hanshin expressway through Kobe was damaged. For the concrete columns, there was inadequate transverse reinforcement, making the columns very weak in shear, causing the longitudinal steel to birdcage and concrete to fail at low stresses. Note lack of cross ties and large spacing of horizontal ties.

# ASCE 61 / POLA Code

- Highly engineered hinge
- Similar to bridge bent




(a) Reinforcement along Pile Length (b) Prototype Class 200 CISS Pile

# D/t or Slenderness Ratio

- Classical (AISC Steel Manual): Compact, Non-Compact, or Slender.
- New (AISC Seismic Provisions for Steel Buildings): Highly Ductile, Moderately Ductile.

Compact

Non-Compact


Slender

# D/t or Slenderness Ratio

- Note thick sections for highly ductile members!
- Note the benefit of filling with concrete!

AISC Steel Manual

**TABLE B4.1 (cont.)**
**Limiting Width-Thickness Ratios for Compression Elements**

| Case | Description of Element | Width Thick-ness Ratio | Limiting Width-Thickness Ratios: $\lambda_p$ (compact) | Limiting Width-Thickness Ratios: $\lambda_r$ (noncompact) | Example |
|---|---|---|---|---|---|
| 14 | Uniform compression in all other stiffened elements | $b/t$ | NA | $1.49\sqrt{E/F_y}$ | |
| 15 | Circular hollow sections | | | | |
| | In uniform compression | $D/t$ | NA | $0.11E/F_y$ | |
| | In flexure | $D/t$ | $0.07E/F_y$ | $0.31E/F_y$ | |

[a] $k_c = \frac{4}{\sqrt{h/t_w}}$, but shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes. (See Cases 2 and 4)
[b] $F_L = 0.7F_y$ for minor-axis bending, major-axis bending of slender-web built-up I-shaped members, and major-axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} \ge 0.7$; $F_L = F_y S_{xt}/S_{xc} \ge 0.5F_y$ for major-axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} < 0.7$. (See Case 2)

AISC 341-16

**TABLE D1.1 (continued)**
**Limiting Width-to-Thickness Ratios for Compression Elements for Moderately Ductile and Highly Ductile Members**

| | Description of Element | Width-to-Thickness Ratio | Limiting Width-to-Thickness Ratio: $\lambda_{hd}$ Highly Ductile Members | Limiting Width-to-Thickness Ratio: $\lambda_{md}$ Moderately Ductile Members | Example |
|---|---|---|---|---|---|
| Stiffened Elements | Webs of built-up box sections used as EBF links | $h/t$ | $0.67\sqrt{\frac{E}{R_yF_y}}$ | $1.75\sqrt{\frac{E}{R_yF_y}}$ | |
| Stiffened Elements | Webs of H-Pile sections | $h/t_w$ | not applicable | $1.57\sqrt{\frac{E}{R_yF_y}}$ | |
| Stiffened Elements | Walls of round HSS | $D/t$ | $0.053\frac{E}{R_yF_y}$ | $0.062\frac{E}{R_yF_y}$ [d] | |
| Composite | Walls of rectangular filled composite members | $b/t$ | $1.48\sqrt{\frac{E}{R_yF_y}}$ | $2.37\sqrt{\frac{E}{R_yF_y}}$ | |
| Composite | Walls of round filled composite members | $D/t$ | $0.085\frac{E}{R_yF_y}$ | $0.17\frac{E}{R_yF_y}$ | |

# Map and Territory

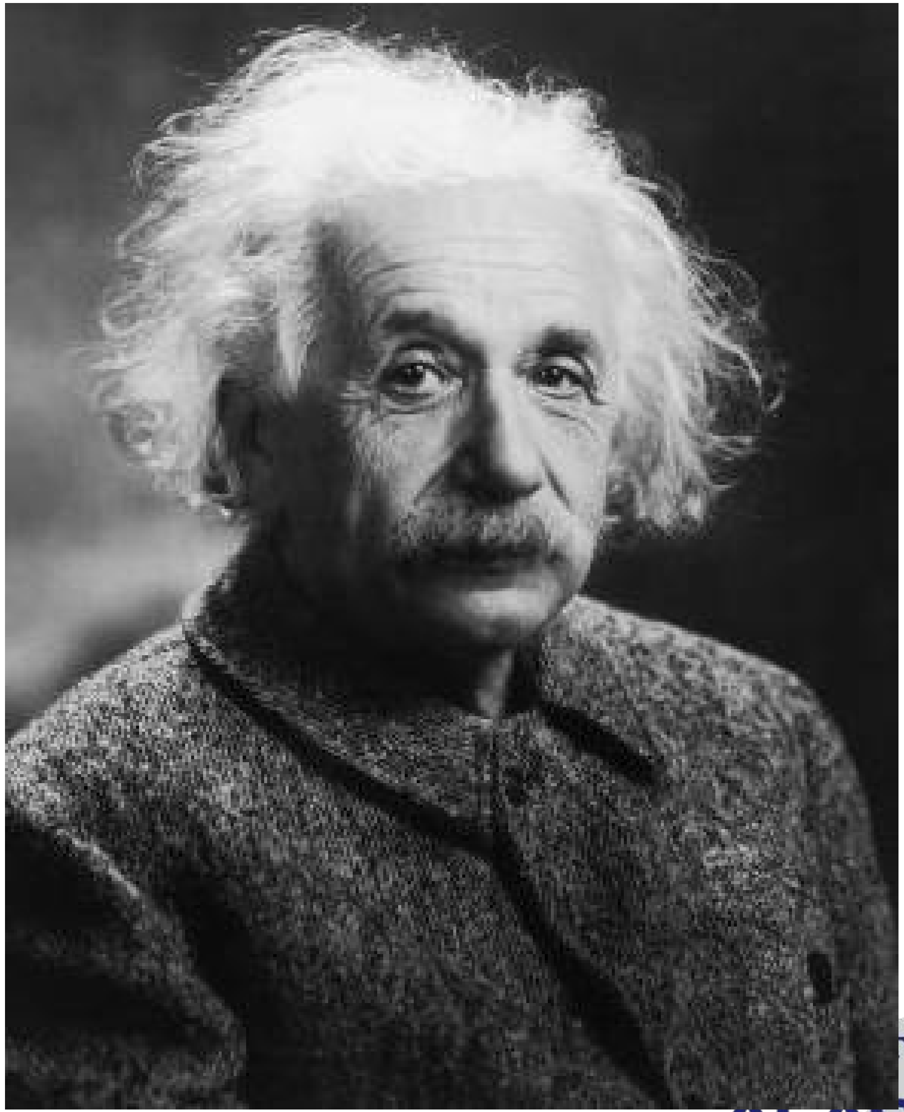
- The Map is Not the Territory
  - 1931 Alfred Korzybski -Polish American scientist / philosopher.

  - The model is not the data
  - All models are wrong (but some are useful)
  - The menu is not the meal

  - Many people do confuse conceptual models with reality
  - Human condition - trying to understand reality

# Map and Territory

- Greatness is providing an accurate map!



# Thank You