Seismic Earth Pressures under Restrained Condition

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C.H.J., Incorporated

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- Seismic Earth Pressure (SEP) – a classic but unresolved problem
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TRB Research Project

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Seismically Induced Earth Pressures for LRFD Seismic Design of Retaining Structures

RESEARCH PROBLEM STATEMENT

In development of the *AASHTO LRFD Bridge Design Specifications* as well as the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (SGS), several unresolved issues associated with the design of earth retaining structures have been identified. The most critical of these unresolved issues are the distribution and magnitude of the seismic earth pressure behind walls that are restrained in their ability to move laterally. There also remains uncertainty regarding seismic earth pressure behind free standing walls that have ability to move laterally during seismic loading. The resolution of these issues is essential to safe and economical seismic design. However, these issues cannot be resolved with the existing knowledge base. Therefore, research is needed to provide a sound basis for these aspects of the seismic design of Earth Retaining Structures (ERS). The results of this work will be useful to design engineers for all types of bridge abutment walls and free standing earth retaining structures in AASHTO Seismic Design Categories C and D, the two highest seismic design categories. It will also provide information on static earth pressures behind restrained wall systems that will be useful to bridge designers in all parts of the country.

ESTIMATE OF PROBLEM FUNDING AND RESEARCH PERIOD

Recommended Funding: $850,000

Research Period: 4 years

Sponsoring Committee: **AFF50, Seismic Design and Performance of Bridges**
Source Info: Edward Kavazanjian, Jr., Associate Professor of Civil Engineering, Arizona State University, Tempe, AZ, 85287-5306; Tel: 480-727-8566; Email: edkavyasu.edu
Date Posted: 07/20/2010
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Index Terms: Earth pressure, Base isolation, Earthquake resistant design, Seismicity, Bridge design, AASHTO LRFD Bridge Design Specifications, Load factor, Bridge abutments, Retaining walls, Geotechnical engineering.

Introduction

- The fact

Although there are many reports of damage to earth retaining walls during earthquakes, almost all of the reports are for either poorly constructed non-engineered walls or walls that failed because of a soil-related failure, with many being in a marine or waterfront environment. Based on a search of literature by the authors, no reports of any damage to building basement walls retaining earth have been found for the 1971 San Fernando, 1987 Whittier Narrows, 1989 Loma Prieta, and 1994 Northridge earthquakes in the United States. Also, reports of damage to building basement walls in foreign earthquakes are few.

  – Lew, M., Sitar, N. and Al Atik, L. (2010). "Seismic Earth Pressures: Fact or Fiction." Proceedings of the Earth Retention Conference 3, Geo-Institute of ASCE, Bellevue, WA.

Introduction

- Main unresolved problems
  - Existing SEP formula may be too conservative
  - SEP distributions – normal triangular, inverted triangular, parabolic
  - Seismic vs. pseudostatic – confusion in pseudostatic seismic coefficient, k_h

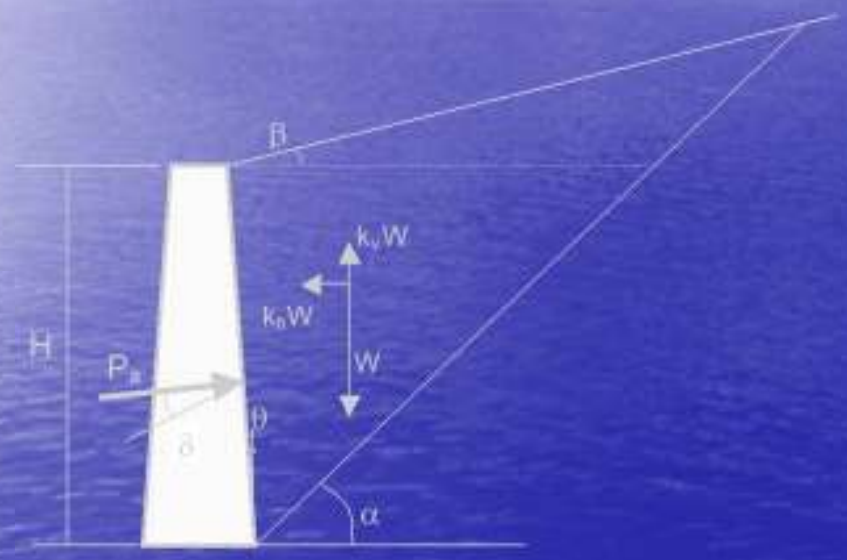
REVIEW OF PREVIOUS STUDIES

Review of Previous Studies

- Cantilever Wall (Flexible / Yielding)
  - Okabe (1924), Mononobe & Matsuo (1929)
  - Seed & Whitman (1970)
  - …
- Restrained Wall (Stiff / Non-yielding)
  - Wood (1973)
  - Ostadan (1998, 2004)
- Most recent
  - Atik & Sitar (2007, 2008, 2010)
  - Maleki & Mahjoubi (2010)

Mononobe-Okabe (M-O) Method

- Okabe (1926), Mononobe & Matsuo (1929)



$$P_{AE} = p_{AE} \cdot \frac{H^2}{2}$$

$$p_{AE} = K_{AE}\gamma(1 - k_v)\cos\delta$$

$$\phi - \beta \geq \psi$$

$$\psi = \tan^{-1}\left(\frac{k_h}{1 - k_v}\right)$$

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

Seed & Whitman (1970)

$$\Delta K_{AE} = \frac{3}{4} k_h \quad \text{or} \quad \frac{\Delta P_{AE}}{\gamma H^2} = \frac{3}{8} k_h$$

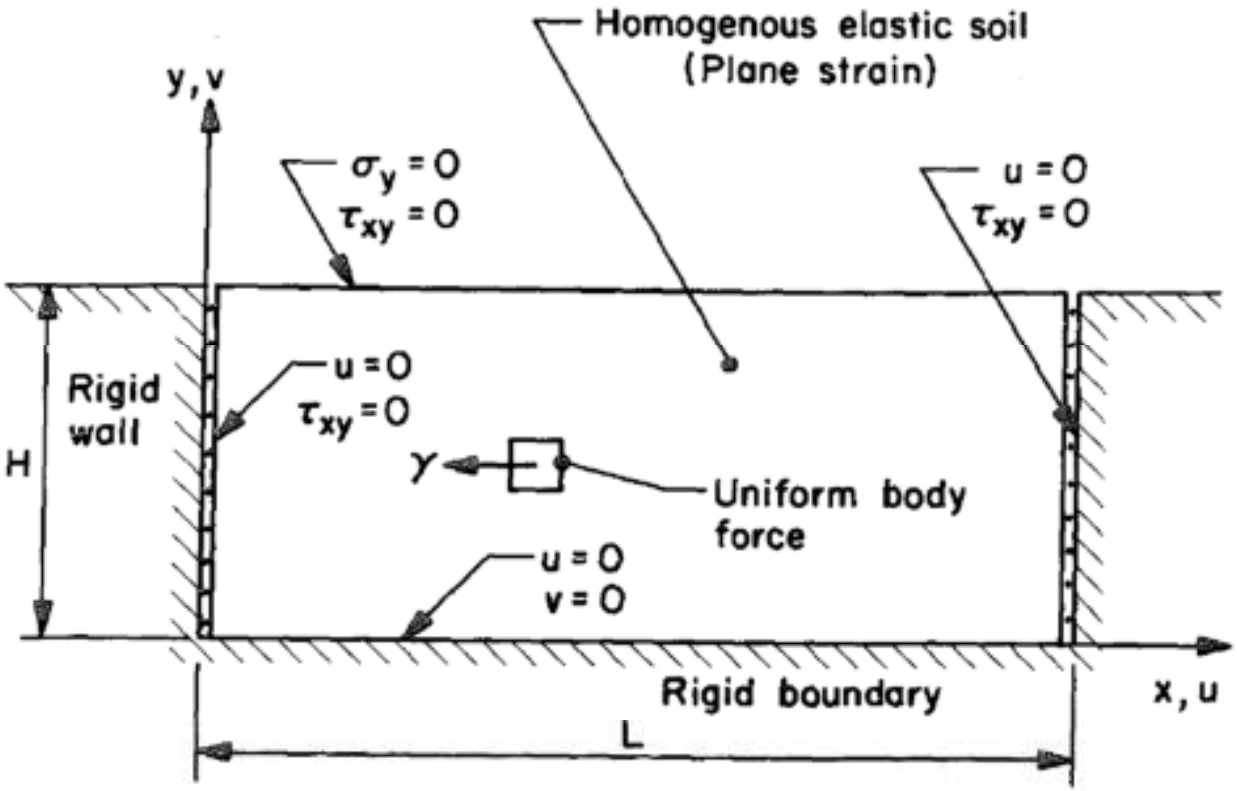
Φ≠35°

k_h	0	0.1	0.2	0.3	0.4
Mononobe-Okabe K_{AE}	0.24	0.29	0.37	0.45	0.58
$K_A + \frac{3}{4} k_h$	0.24	0.31	0.39	0.46	0.54

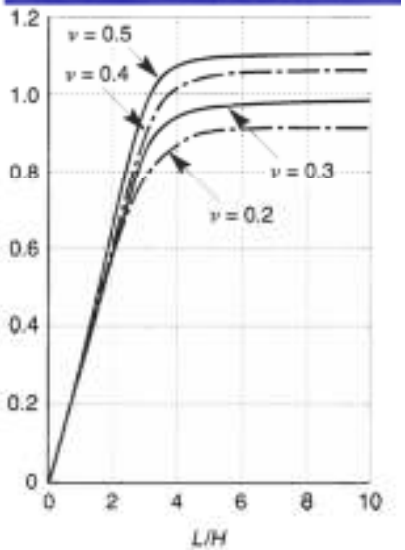
Φ=38°

Seed & Whitman (1970)						
	k_h	0	0.1	0.2	0.3	0.4
Φ=30°	M-O K_{AE}	0.33	0.4	0.49	0.63	0.89
	Ka+3/4k_h	0.33	0.405	0.48	0.555	0.63
Φ=33°	M-O K_{AE}	0.29	0.36	0.44	0.57	0.78
	Ka+3/4k_h	0.29	0.365	0.44	0.515	0.59
Φ=35°	M-O K_{AE}	0.27	0.33	0.41	0.53	0.72
	Ka+3/4k_h	0.27	0.345	0.42	0.495	0.57
Φ=38°	M-O K_{AE}	0.24	0.29	0.37	0.48	0.58
	Ka+3/4k_h	0.24	0.31	0.39	0.46	0.54
by Fred Yi, 2/16/2011						

Wood (1973)



RIGID WALL PROBLEM



$$\Delta P_{eq} = \gamma H^2 \frac{a_h}{g} F_p$$

Ostadan (1998, 2004)

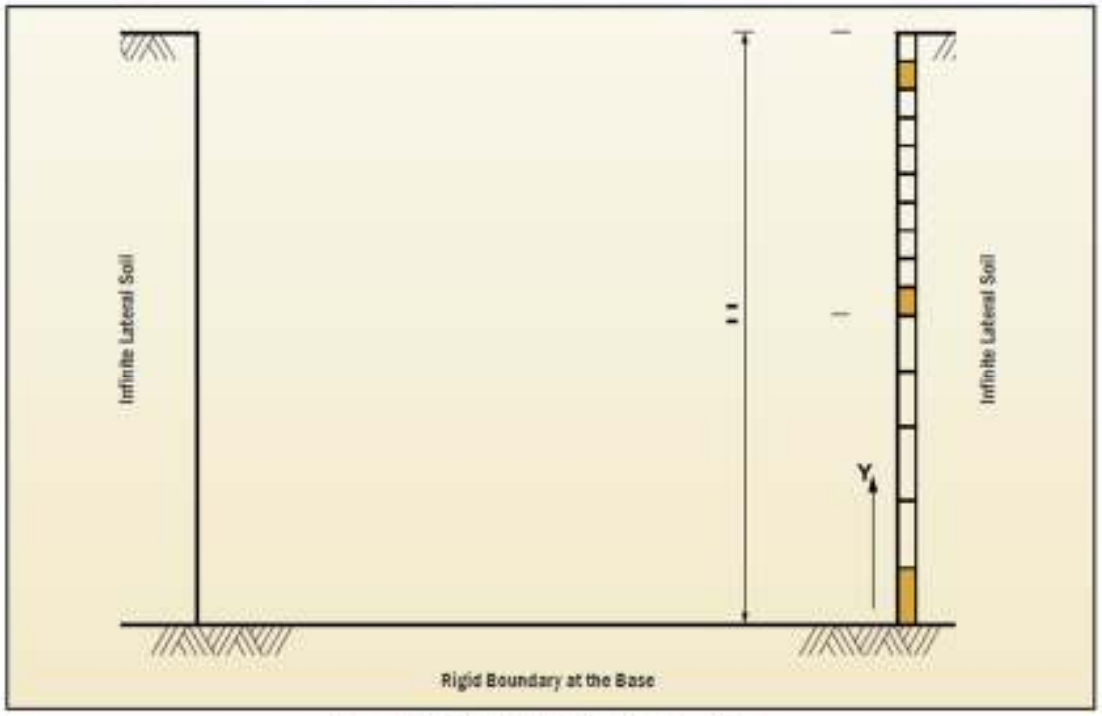


Figure 1. Typical SASSI Model of the Foundation

$$p(y)=-.0015+5.05y-15.84y^2+28.25y^3-24.59y^4+8.14y^5 \quad (3)$$

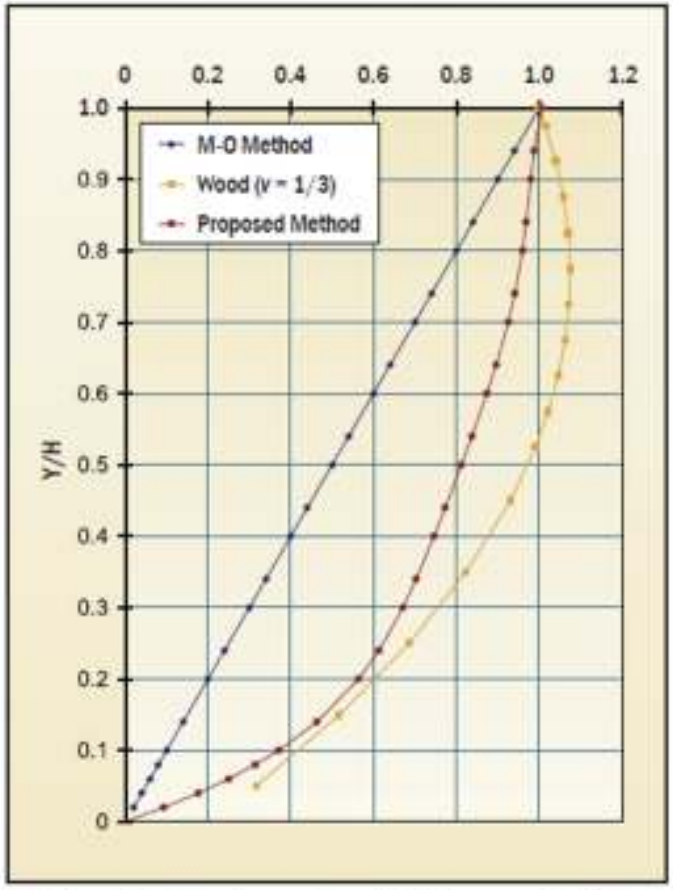


Figure 4. Comparison of Normalized Pressure Profiles

Atik & Sitar (2007, 2008, 2010)

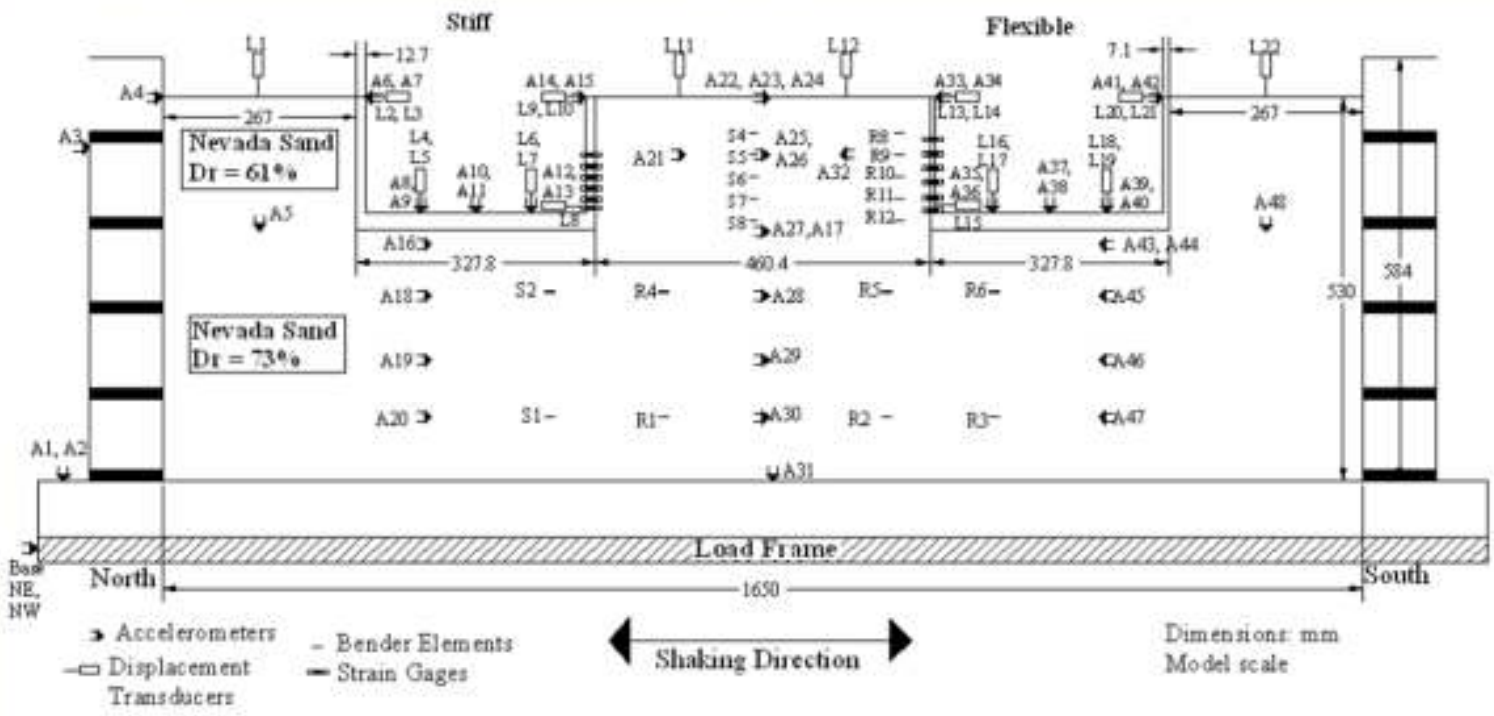
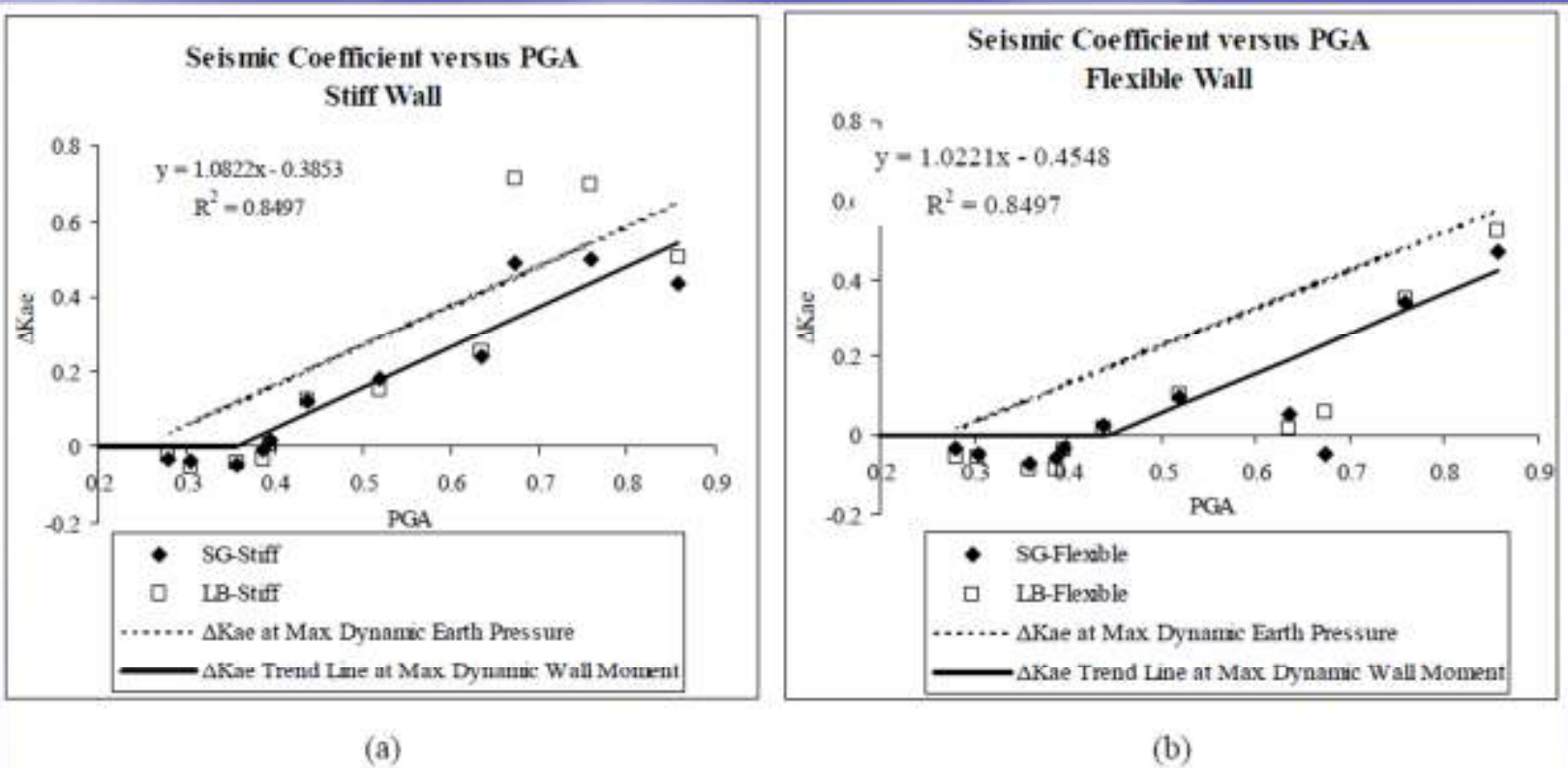


Fig. 3.2 LAA01 model configuration, profile view.

Atik & Sitar (2007, 2008, 2010)



(a) (b)

Fig. 6.17 Back-calculated dynamic earth pressure coefficients at time of maximum dynamic wall moments and maximum dynamic earth pressures on stiff and flexible walls as function of peak ground acceleration measured at top of soil in free field.

Maleki & Mahjoubi (2010)

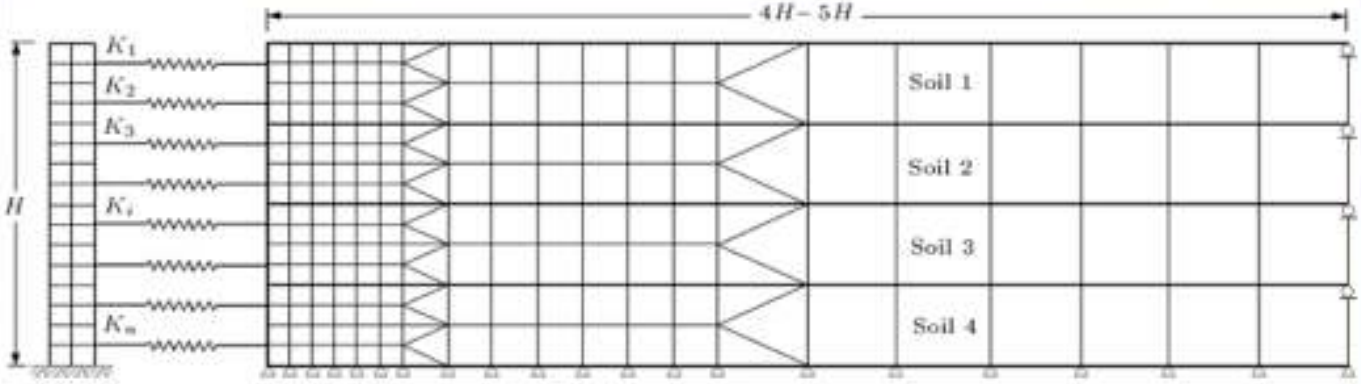


Figure 3. Soil-wall finite element model.

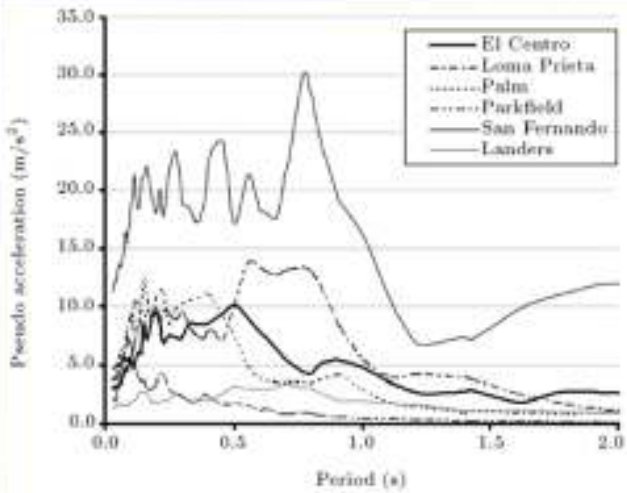


Figure 4. Pseudo acceleration (PSA) spectra for selected ground motions.

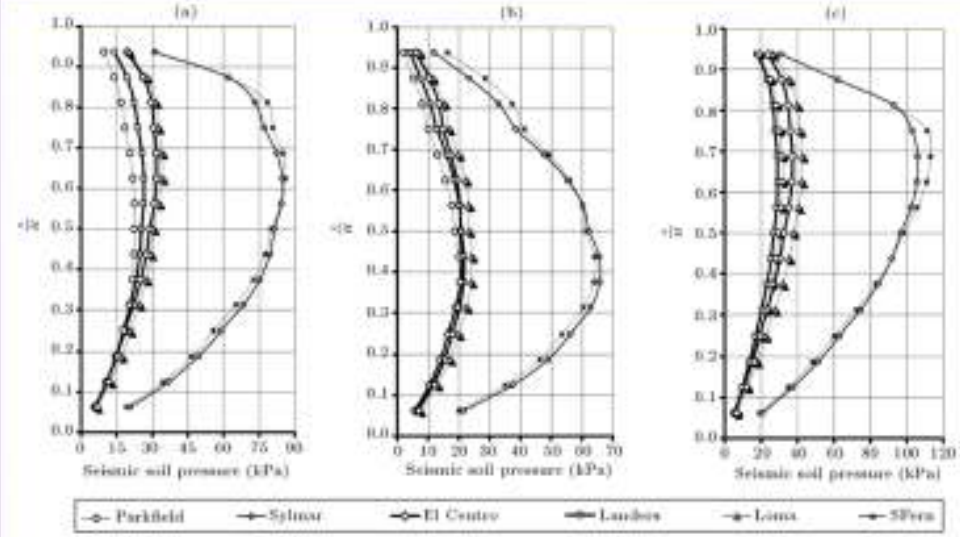


Figure 7. Seismic soil pressure for 8 m height wall. (a) Bridge abutment; (b) flexible wall; (c) rigid wall.

Purpose of This Study

- This study intends to solve
  - Effects of soil strength parameters
  - Effects of base conditions
  - Relationship between k_H and PGA
  - A simple equation for engineers

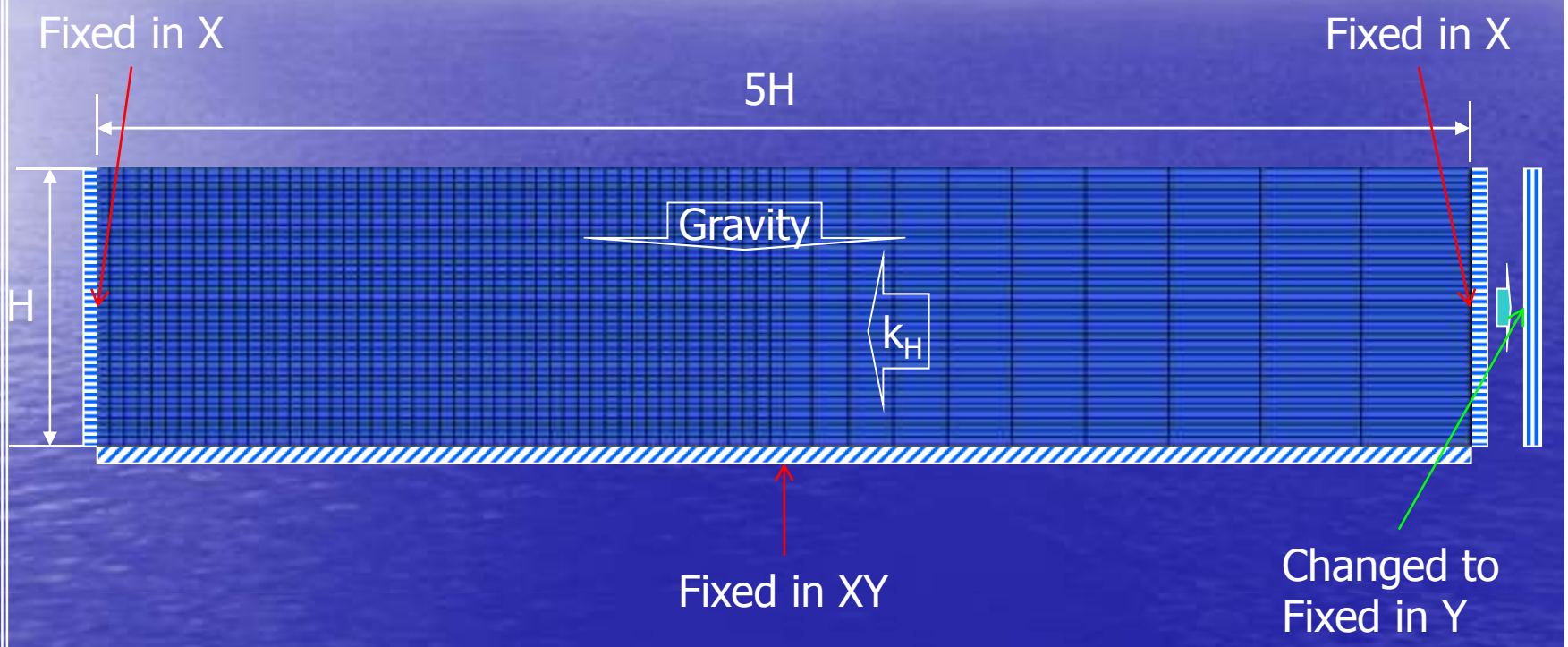
PSEUDOSTATIC NUMERICAL SIMULATION

Pseudostatic Simulation by FEM

- Soil property
  - Elastoplastic
  - Mohr-Coulomb failure criteria (c=0, Φ)
- Boundary conditions
  - Rigid base model
    - Wall movement = 0 & deflection = 0
  - Non-rigid base model
    - Wall movement & deflection = 0
    - Wall movement ≠ 0, deflection = 0

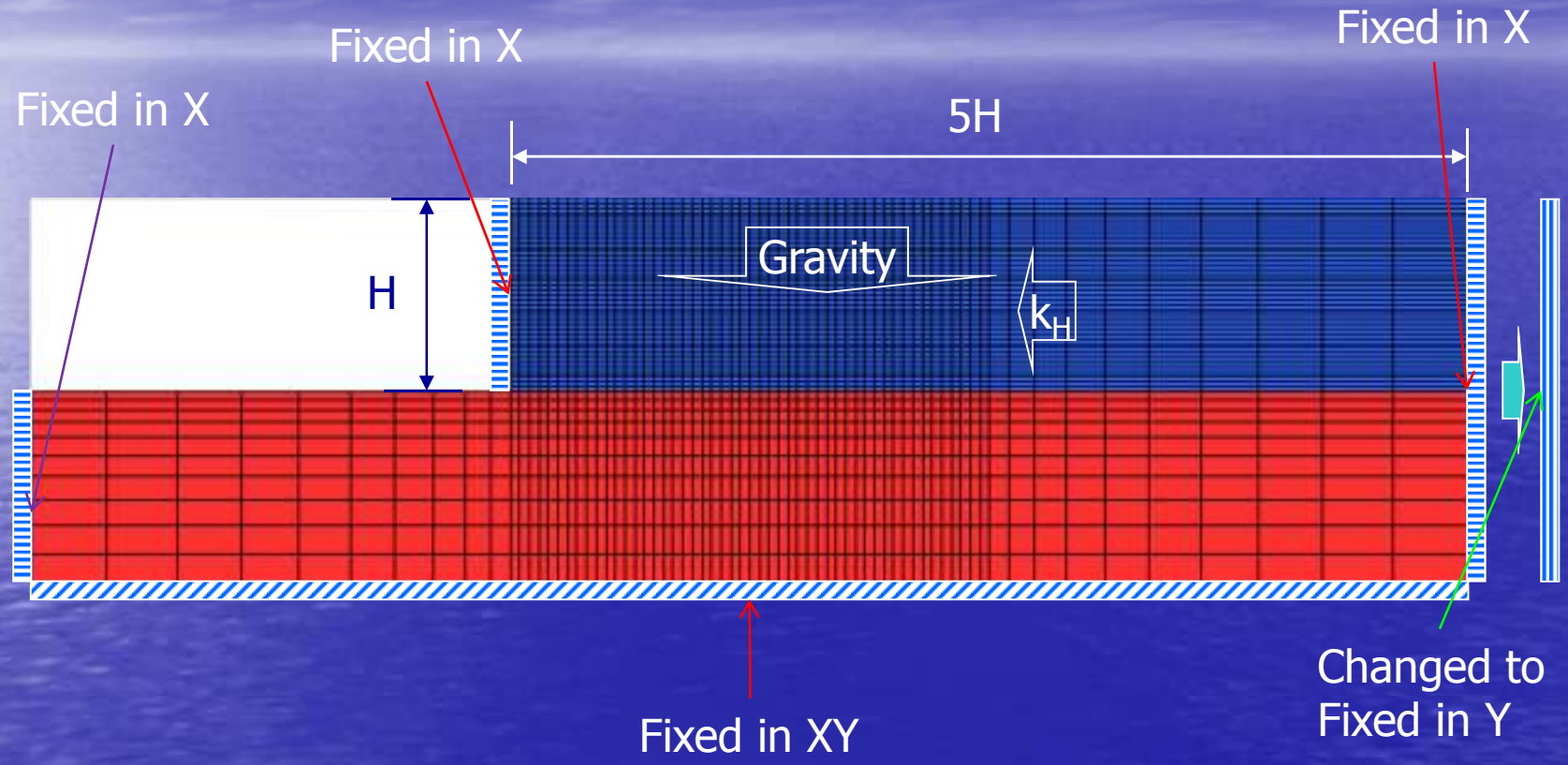
FEM model

- Rigid base, Restrained Rigid Wall



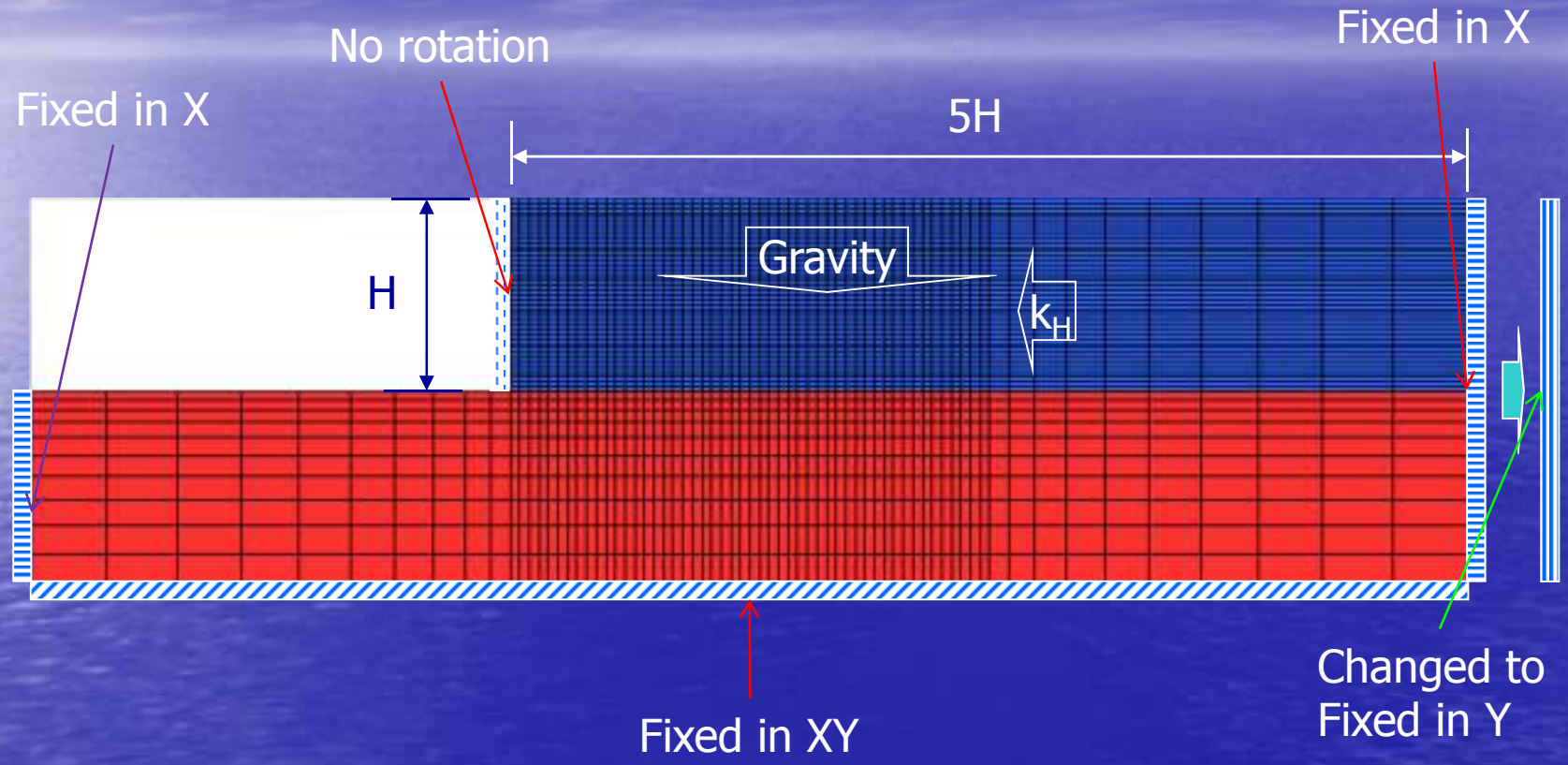
FEM model

- Non-Rigid base, Restrained Rigid Wall



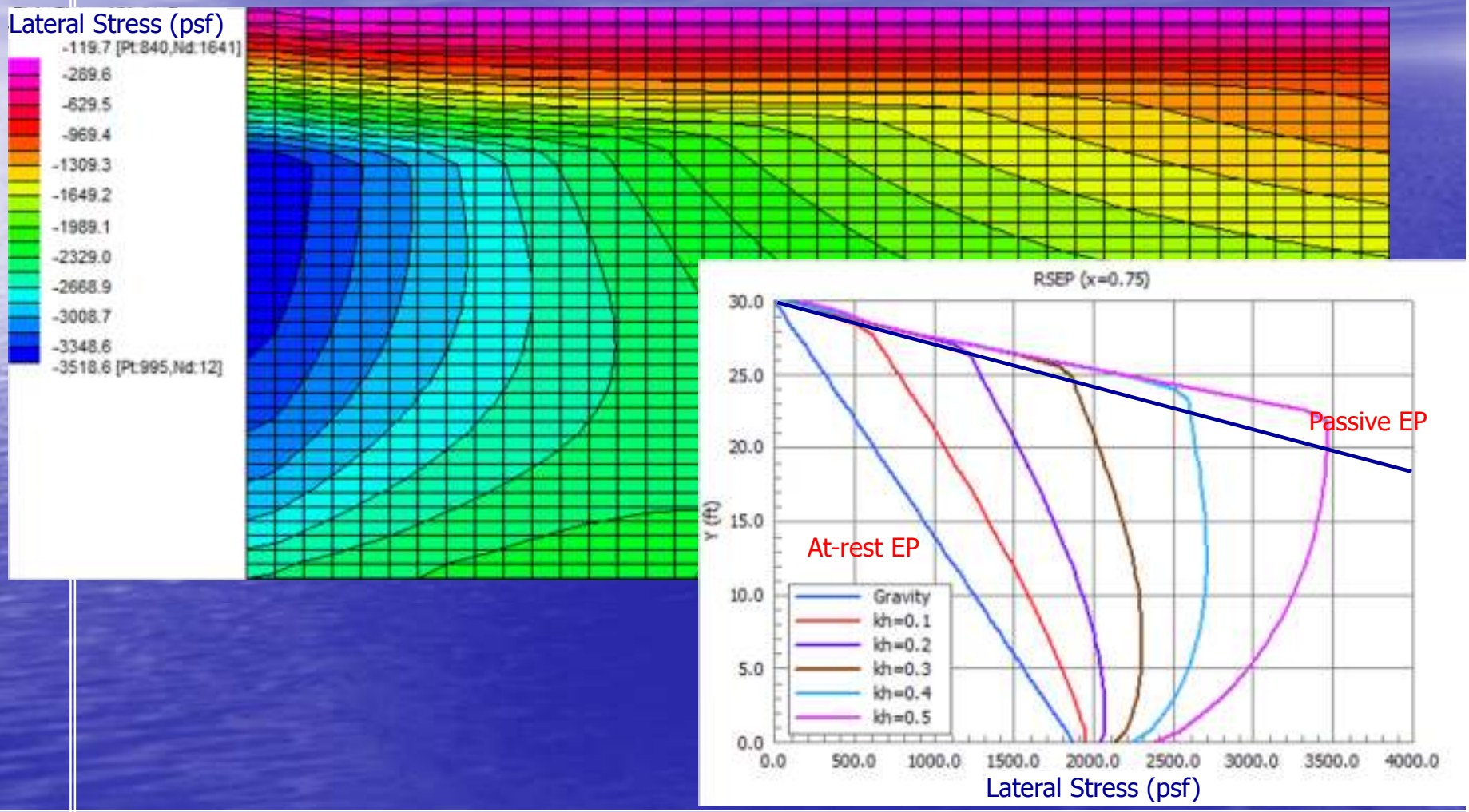
FEM model

- Non-Rigid base, Rigid Wall



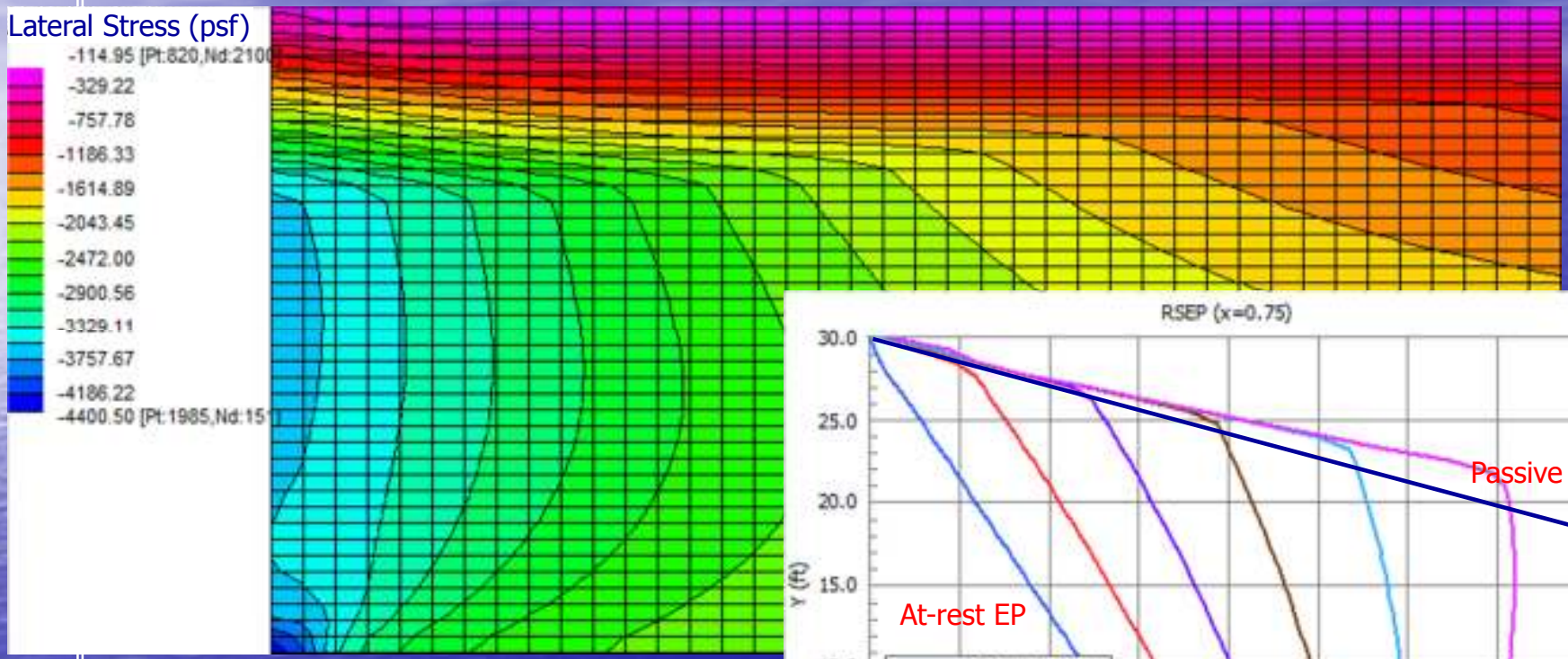
FEM Results

- Rigid base, Restrained Rigid Wall



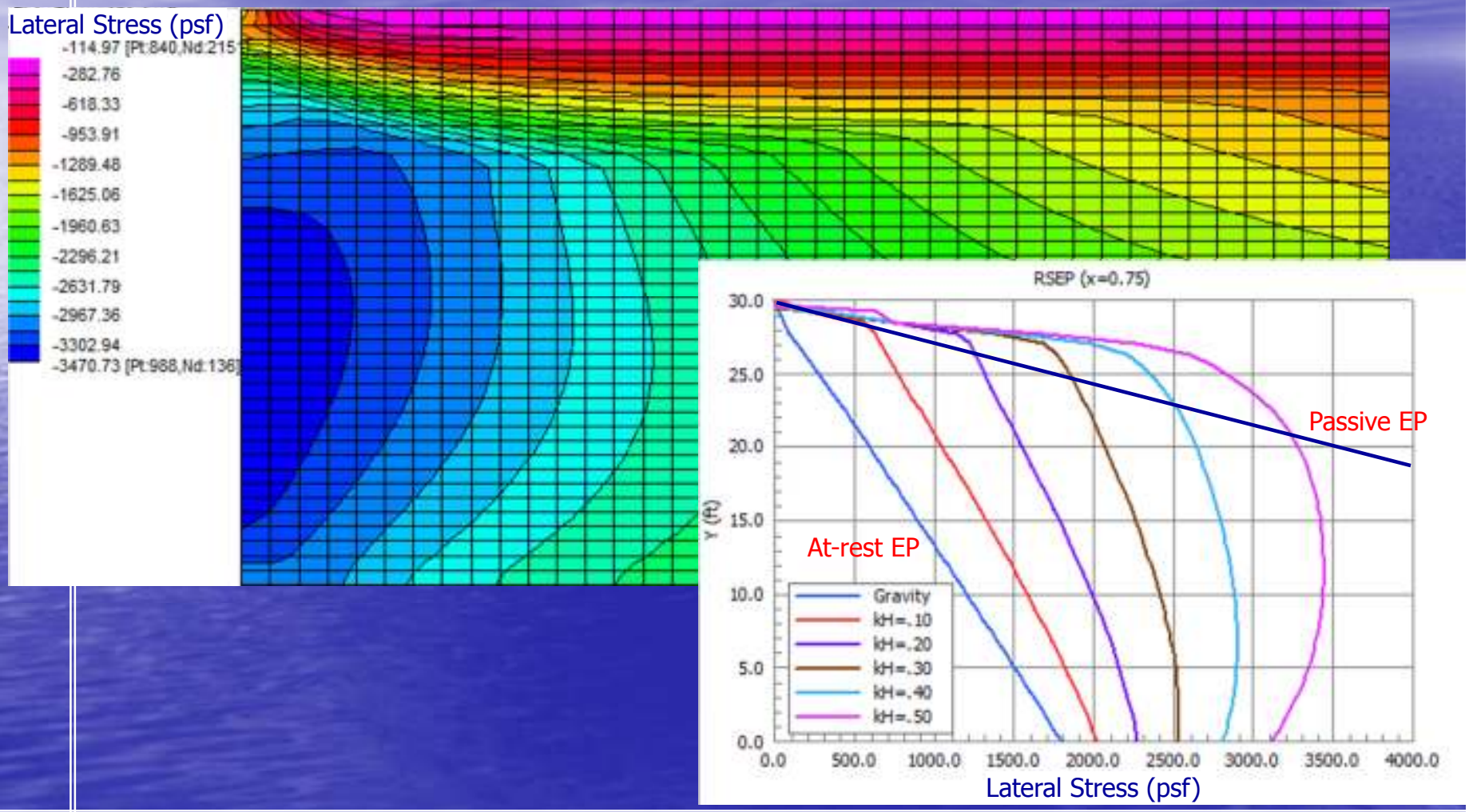
FEM Results

- Non-Rigid base, Restrained Rigid Wall



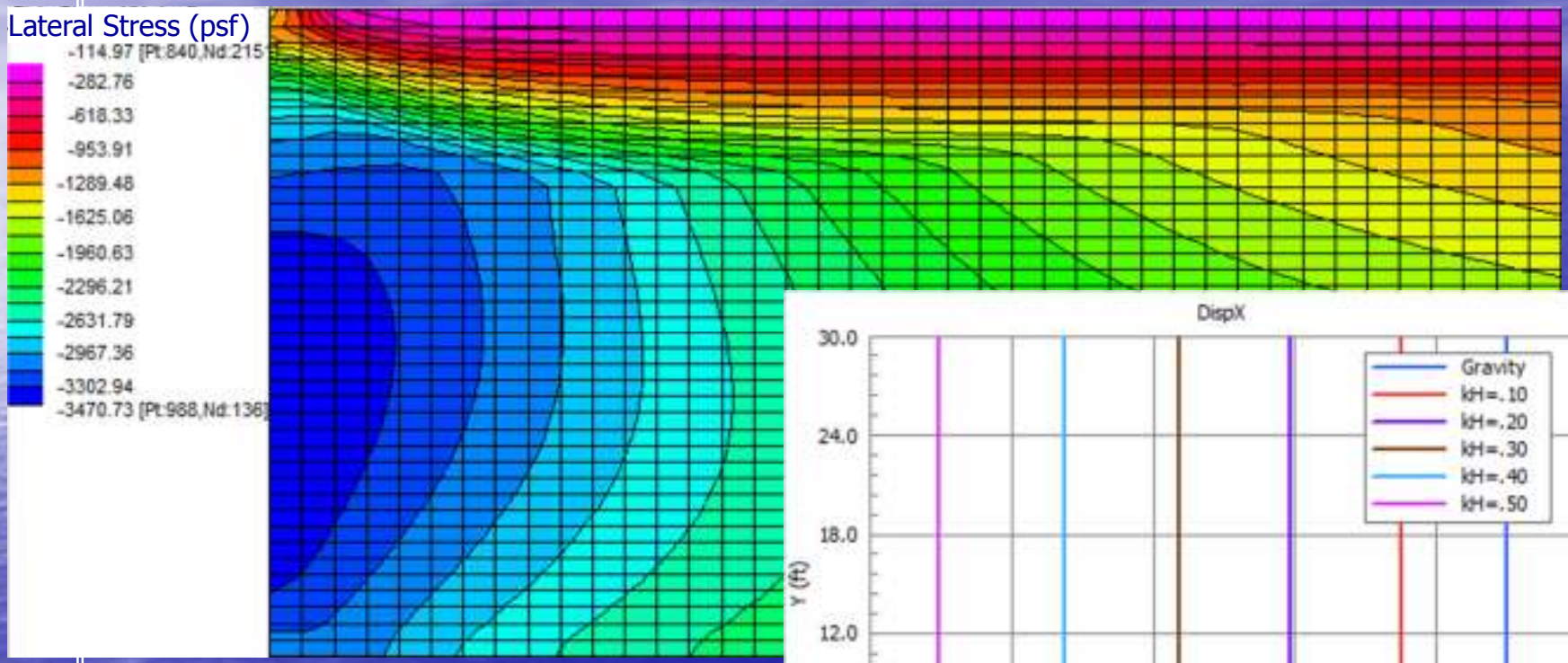
FEM Results

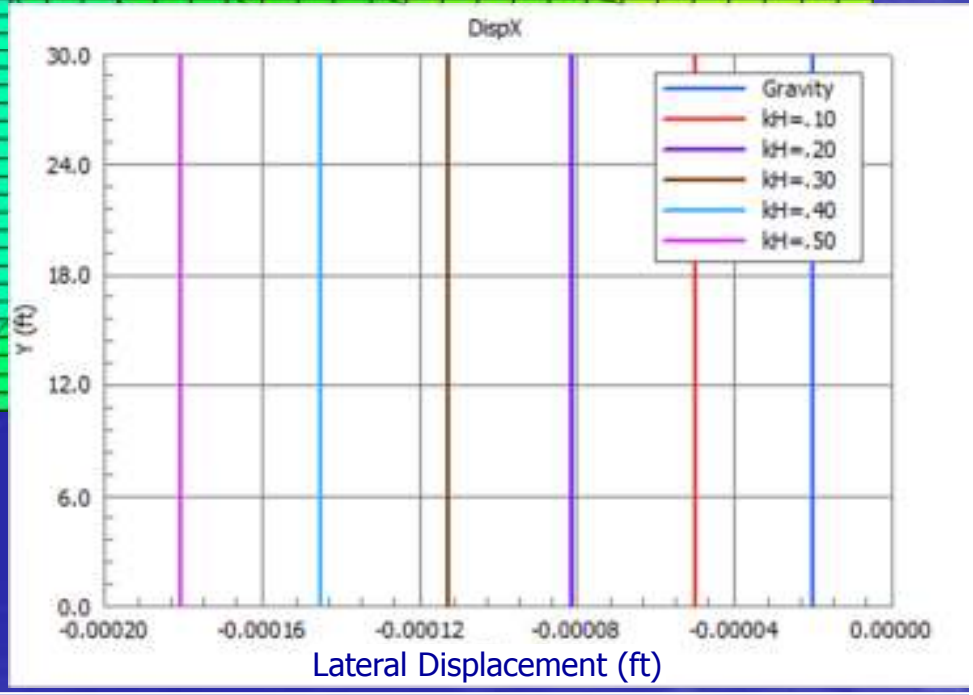
- Non-Rigid base, Rigid Wall



FEM Results

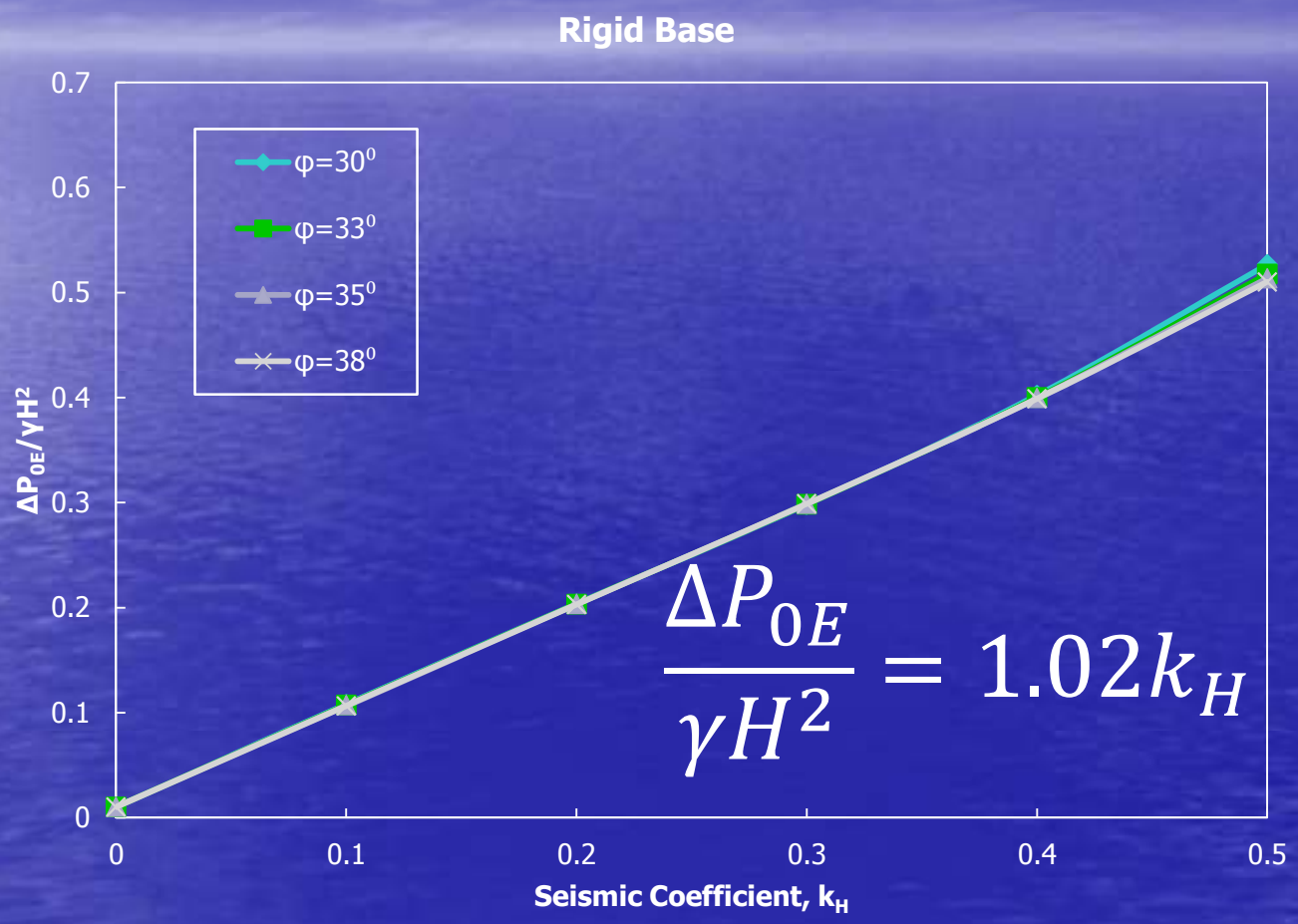
- Non-Rigid base, Rigid Wall





Seismic Earth Pressure

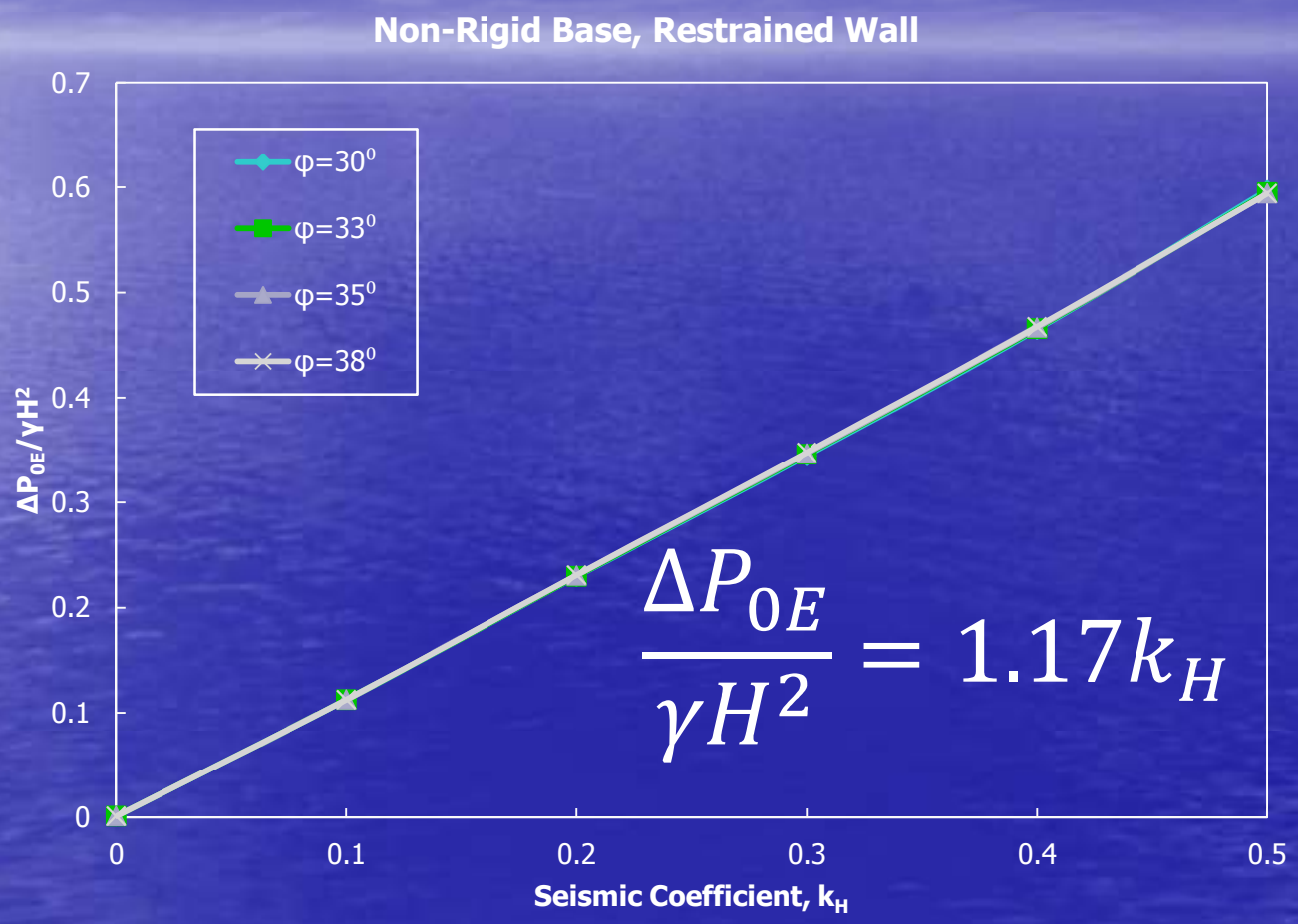
- Rigid base, Restrained Rigid Wall



$$\frac{\Delta P_{0E}}{\gamma H^2} = 1.02 k_H$$

Seismic Earth Pressure

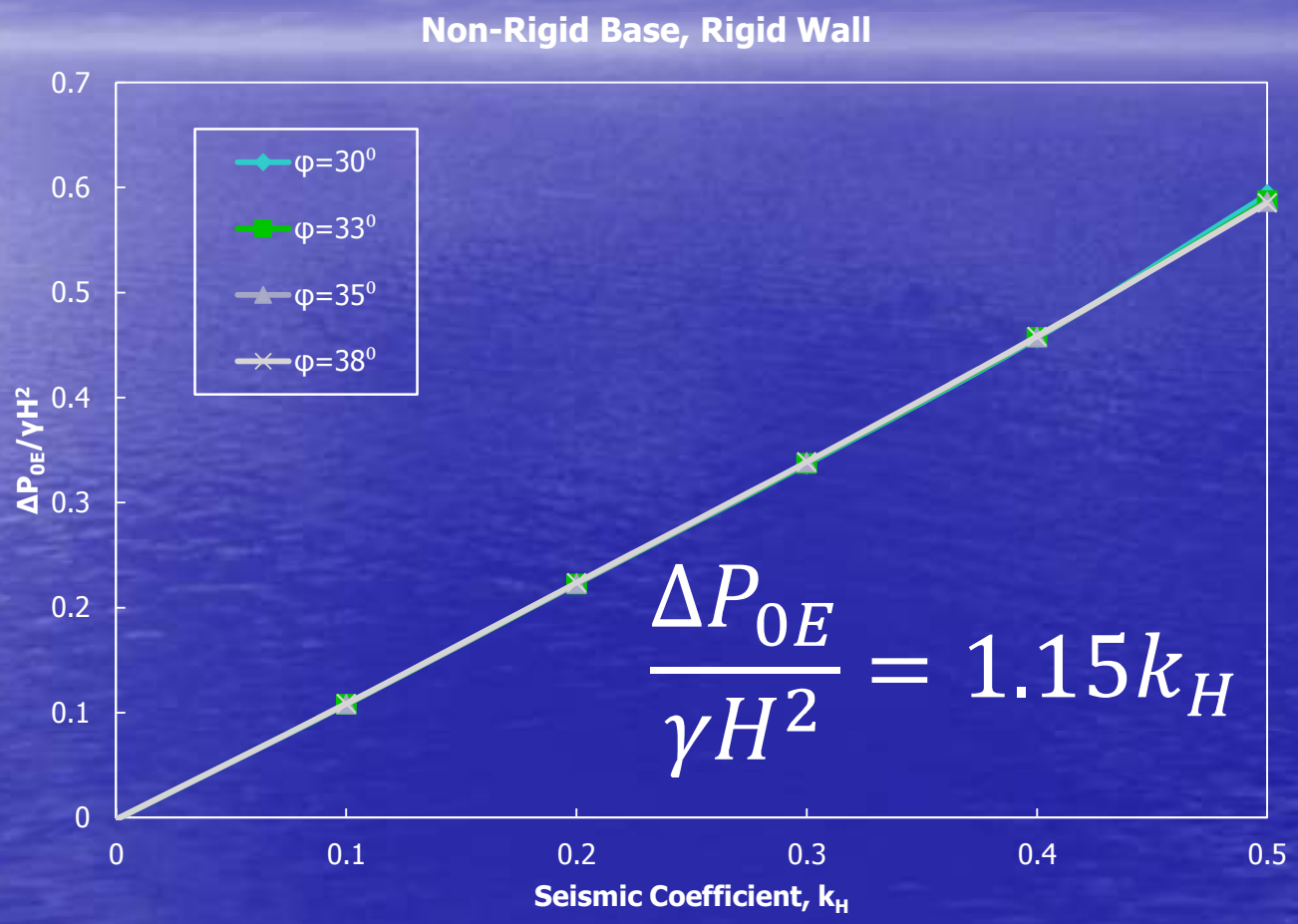
- Non-Rigid base, Restrained Rigid Wall



$$\frac{\Delta P_{0E}}{\gamma H^2} = 1.17 k_H$$

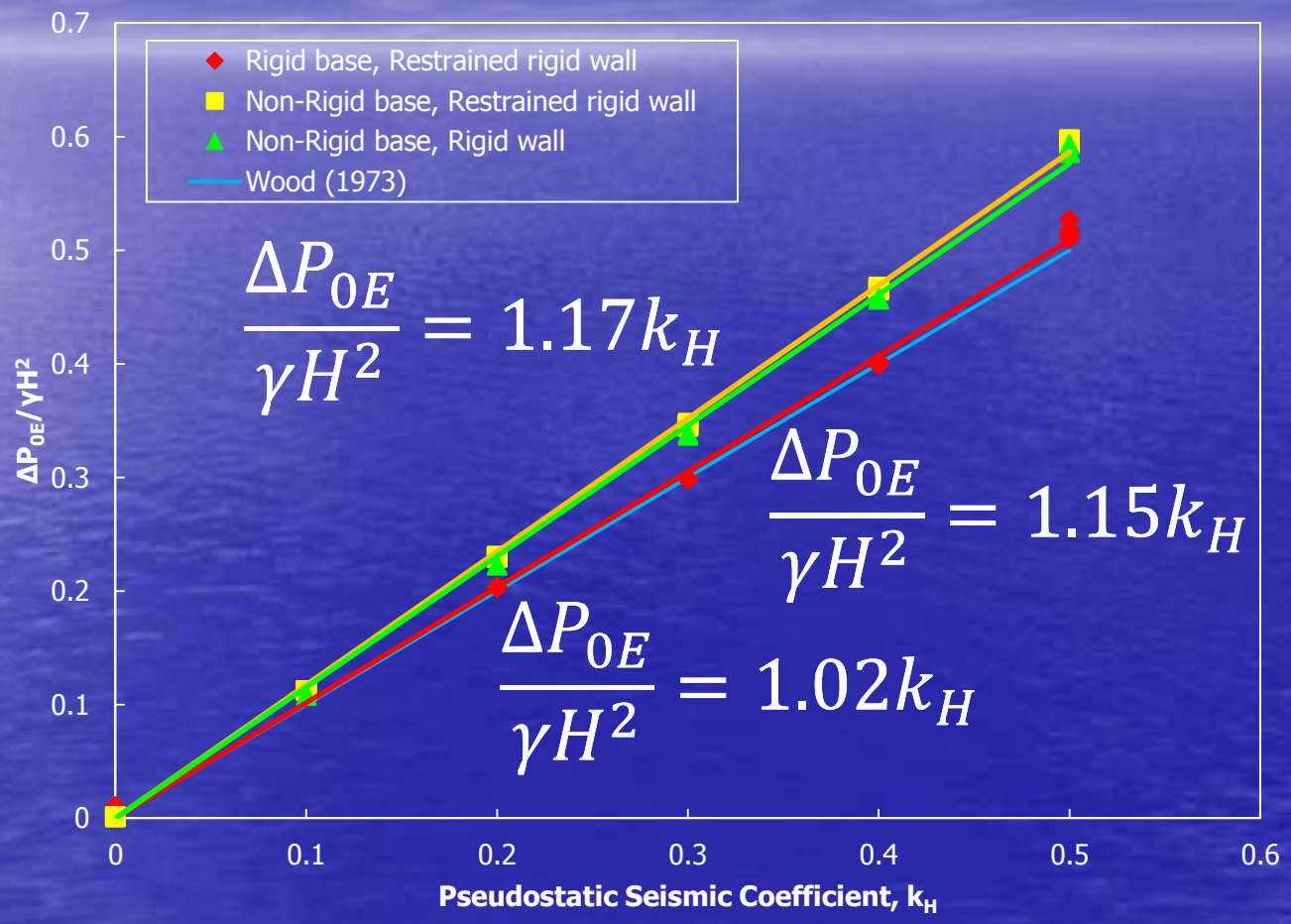
Seismic Earth Pressure

- Non-Rigid base, Rigid Wall



$$\frac{\Delta P_{0E}}{\gamma H^2} = 1.15 k_H$$

Seismic Earth Pressure



Seismic Earth Pressures of Rigid Walls

- Rigid base, Restrained Rigid Wall

$$\frac{\Delta P_{0E}}{\gamma H^2} = 1.02 k_H$$

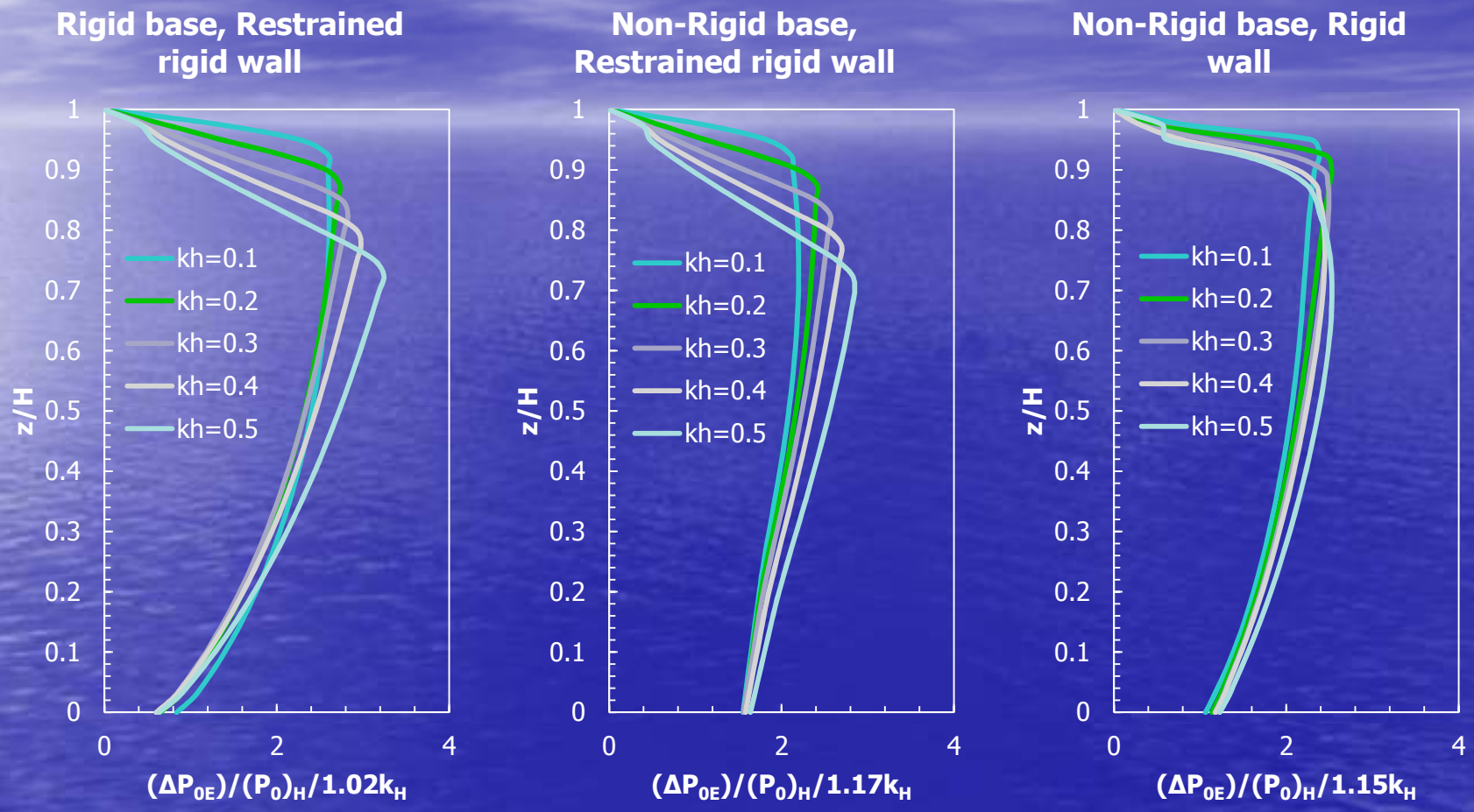
- Non-Rigid base, Restrained Rigid Wall

$$\frac{\Delta P_{0E}}{\gamma H^2} = 1.17 k_H$$

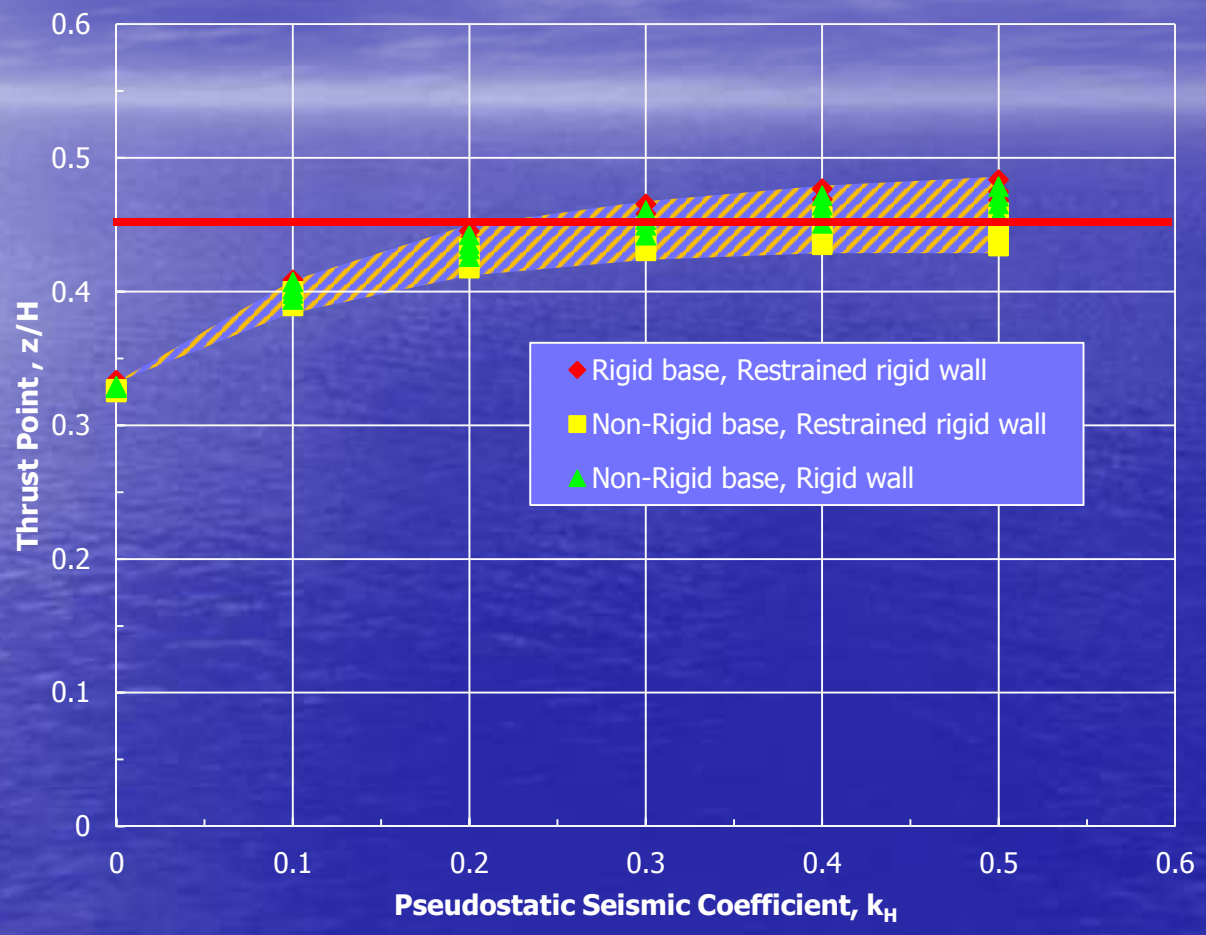
- Non-Rigid base, Rigid Wall

$$\frac{\Delta P_{0E}}{\gamma H^2} = 1.15 k_H$$

Distribution of Seismic Earth Pressure



Thrust Point of Seismic Earth Pressure



PSEUDOSTATIC SEISMIC COEFFICIENT, k_h

Seismic Coefficient (k_H)

- Confusion:
  - k_H=PGA
    - Seed & Whitman (1970)
    - FEMA 450 ($k_H=S_{DS}/2.5$)
    - HCHRP Report 611
    - FHWA-NHI-10-024
  - k_H=PGA/2
    - ASSHTO LRFD Bridge Design Specification, 2010
    - Kramer (1996), k_H=(1/3 ~ ½) PGA
- What should it be?

What should k_h be?

- The magic number of 0.65

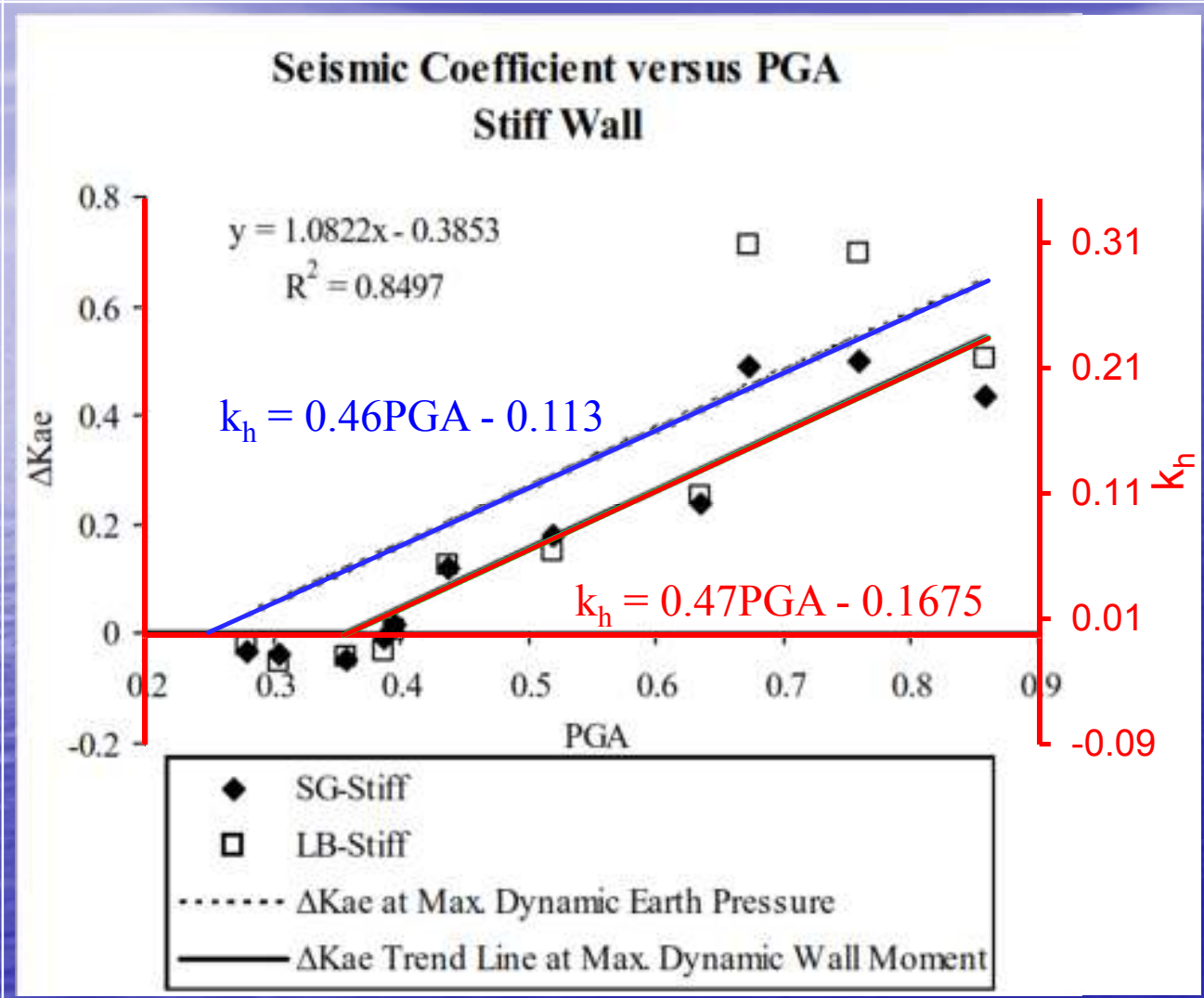
Consequently, Seed and Idriss (1971) chose to represent earthquake-induced cyclic stresses by using a representative value (or equivalent uniform value) equal to 65% of the peak cyclic stress. The corresponding earthquake-induced CSR is therefore computed as

$$\mathrm{CSR} = 0.65\frac{\tau_{\max}}{\sigma'_{vc}} = 0.65\frac{\sigma_{vc}}{\sigma'_{vc}}\frac{a_{\max}}{g}r_d \qquad (25)$$

The choice of 0.65 to represent a reference stress level is somewhat arbitrary, but it was selected in the beginning of the development of liquefaction evaluation procedures in 1966 and has been in use ever since. More importantly, the overall liquefaction evaluation procedure would be essentially unaffected by the choice of a different reference stress ratio, provided that the adjustment factors for the duration of shaking and the empirically derived liquefaction correlations were all derived for that reference stress (see Section 3.5).

Idriss & Boulanger (2008), Soil Liquefaction During Earthquakes, EERI MNO-12

Data by Atik & Sitar (2008)

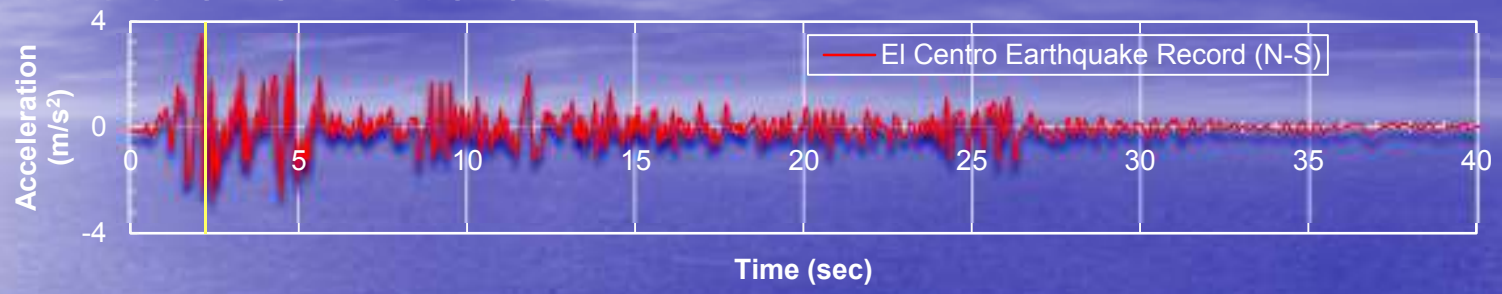


PGA	ΔK_{ae}	k_H
0.4	0.05	0.02
0.5	0.16	0.07
0.6	0.26	0.11
0.7	0.37	0.16
0.8	0.48	0.21

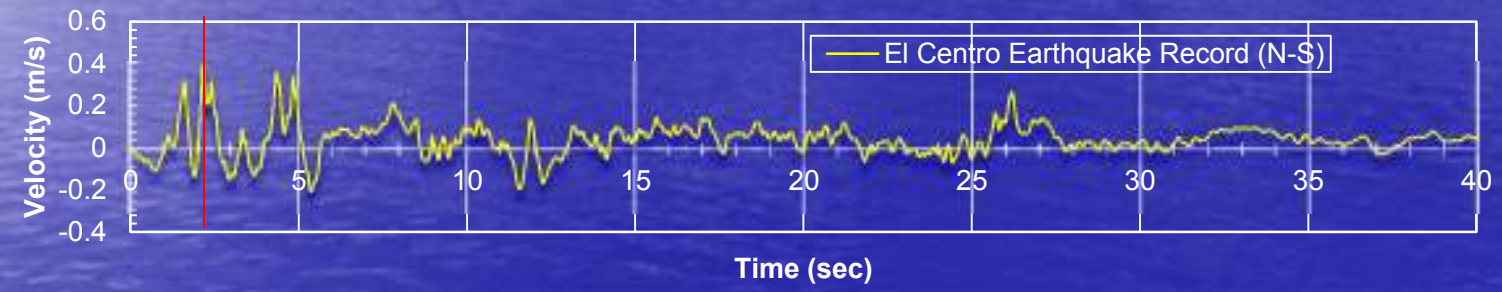
Atik & Sitar (2008), Experimental and Analytical Study of the Seismic Performance of Retaining Structures, University of California, Berkeley

Examination of an earthquake record

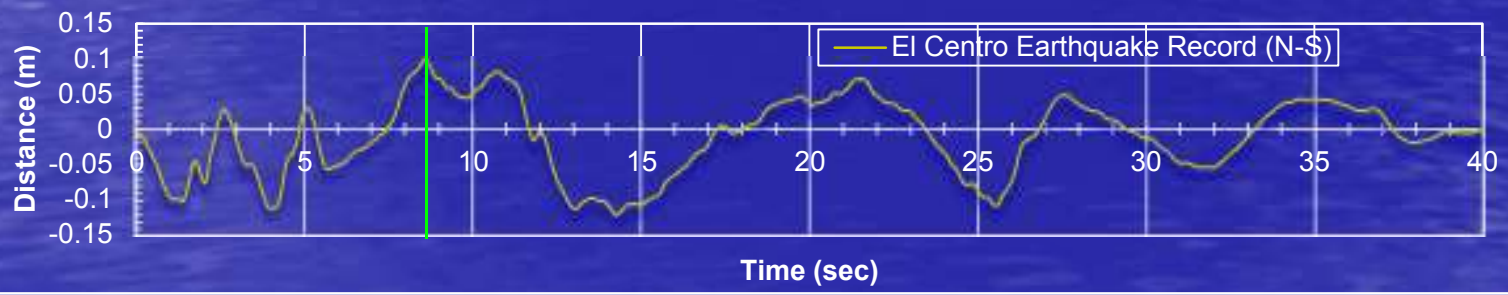
- Acceleration records



- Velocity time history

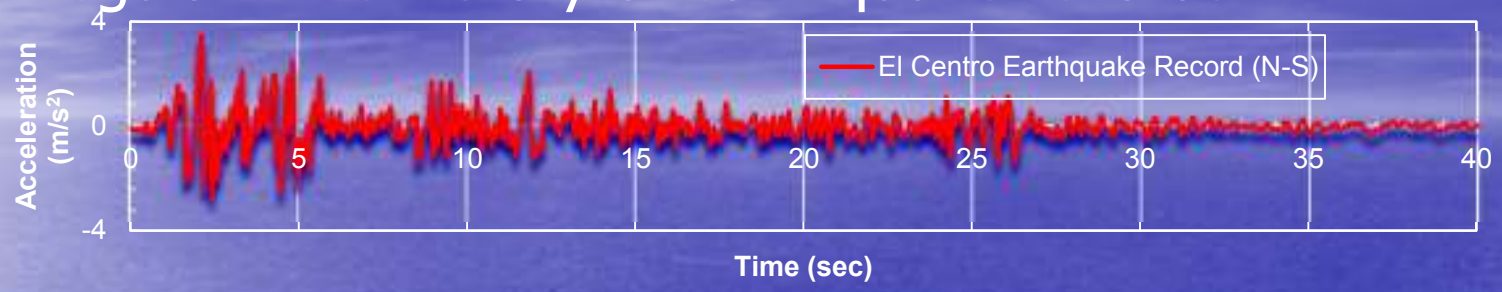


- Displacement time history

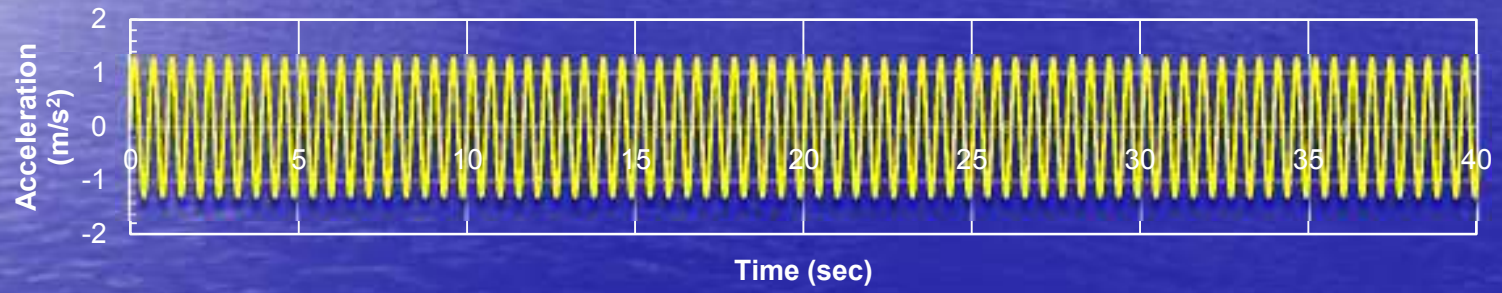


Earthquake record, cyclic and pseudostatic

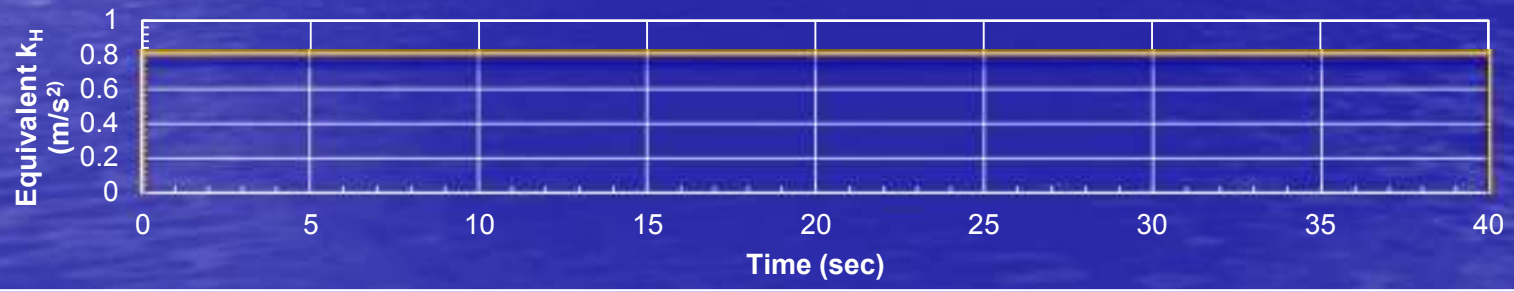
- Irregular time history of earthquake records



- Cyclic loading in laboratory testing

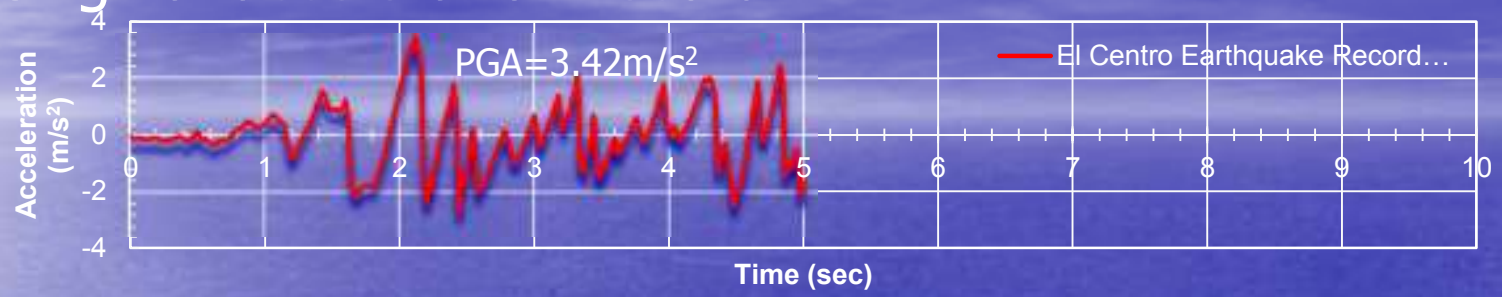


- Pseudostatic seismic coefficient

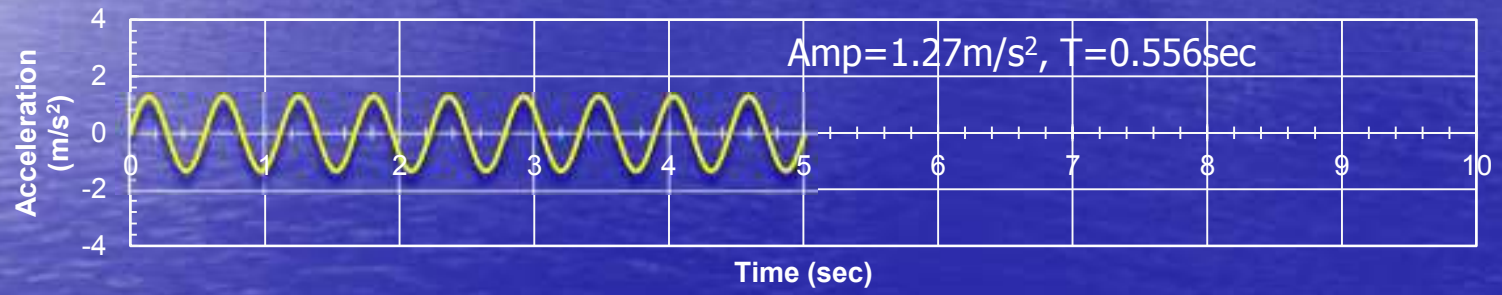


Considering Energy Conservation: t=0~5 s

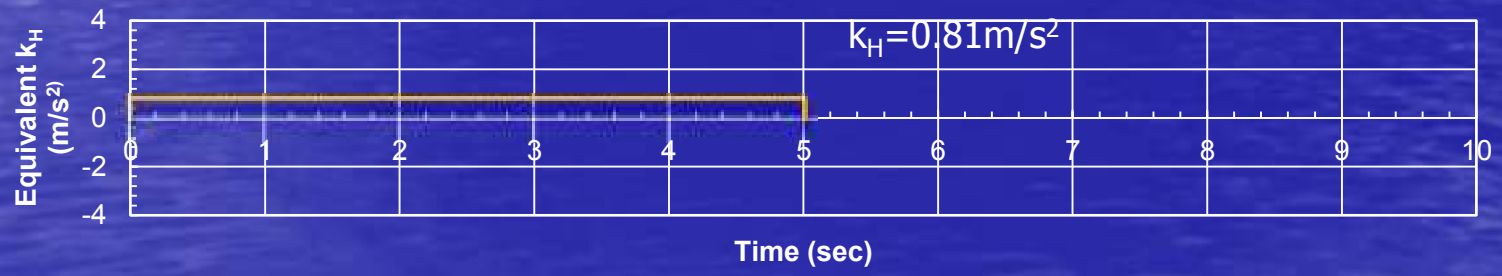
- Original acceleration record



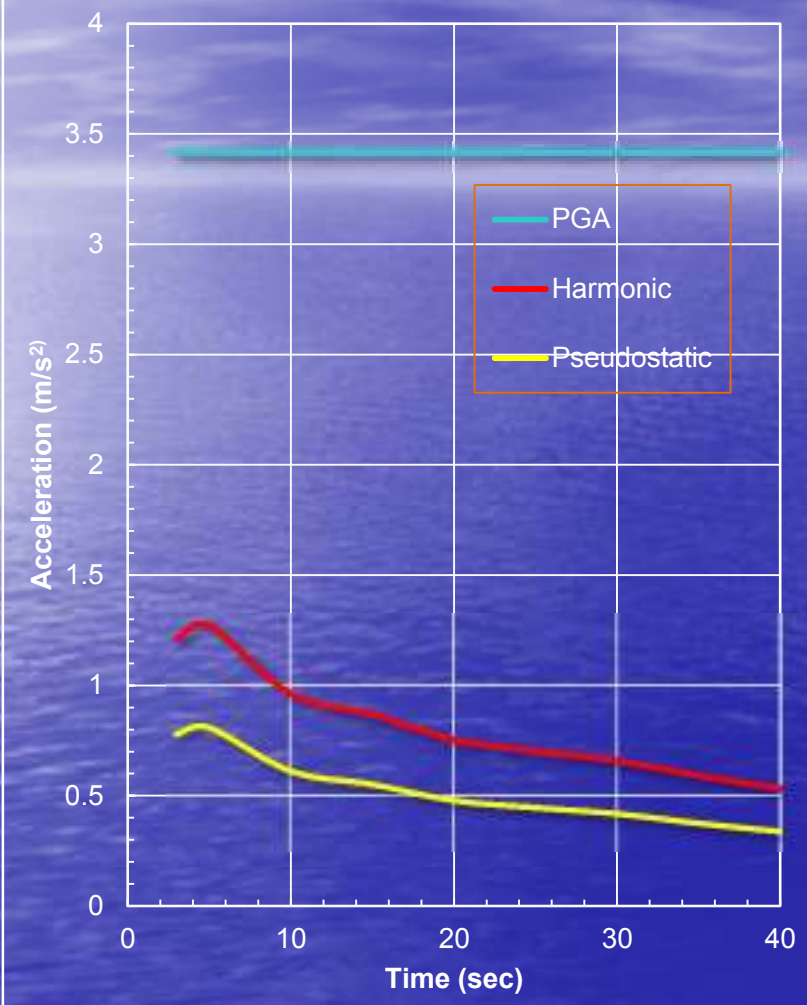
- Equivalent Harmonic cyclic wave

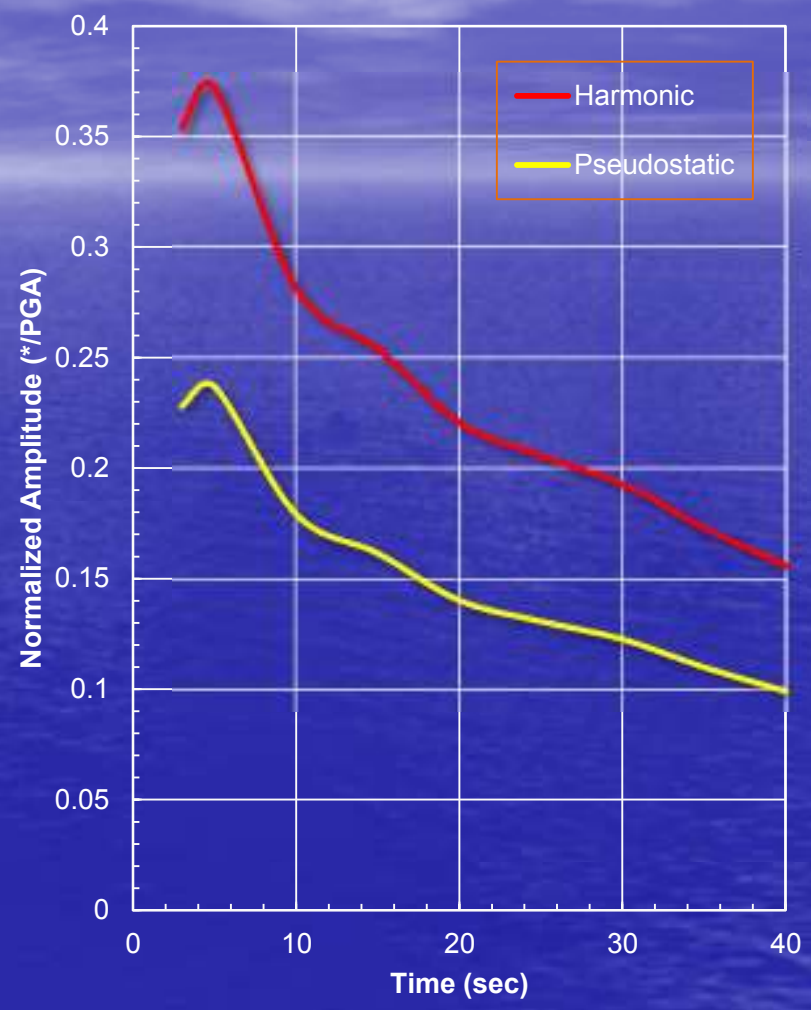


- Equivalent Pseudostatic load



Equivalent cyclic wave Amp & Pseudostatic k_H

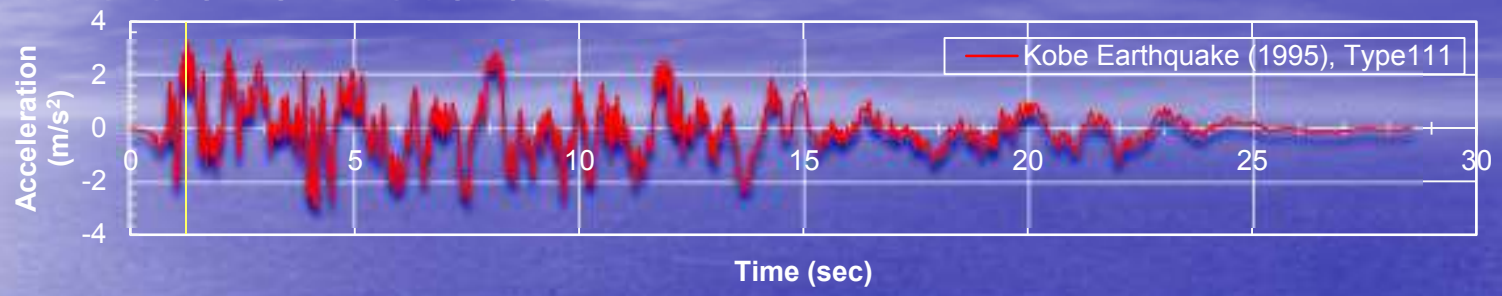




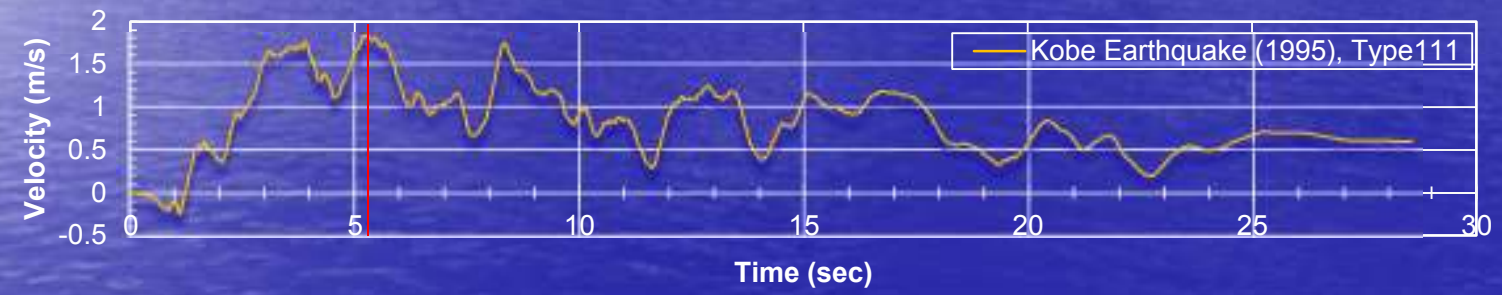
El Centro Earthquake Record (N-S)

Examination of an earthquake record

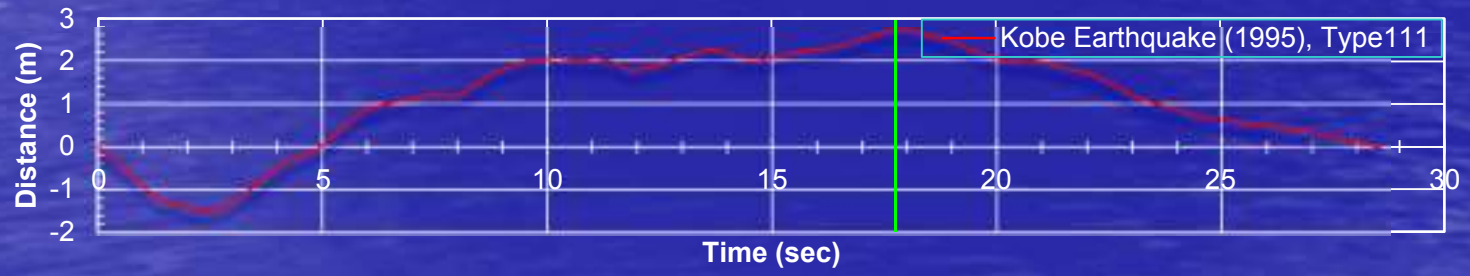
- Acceleration records



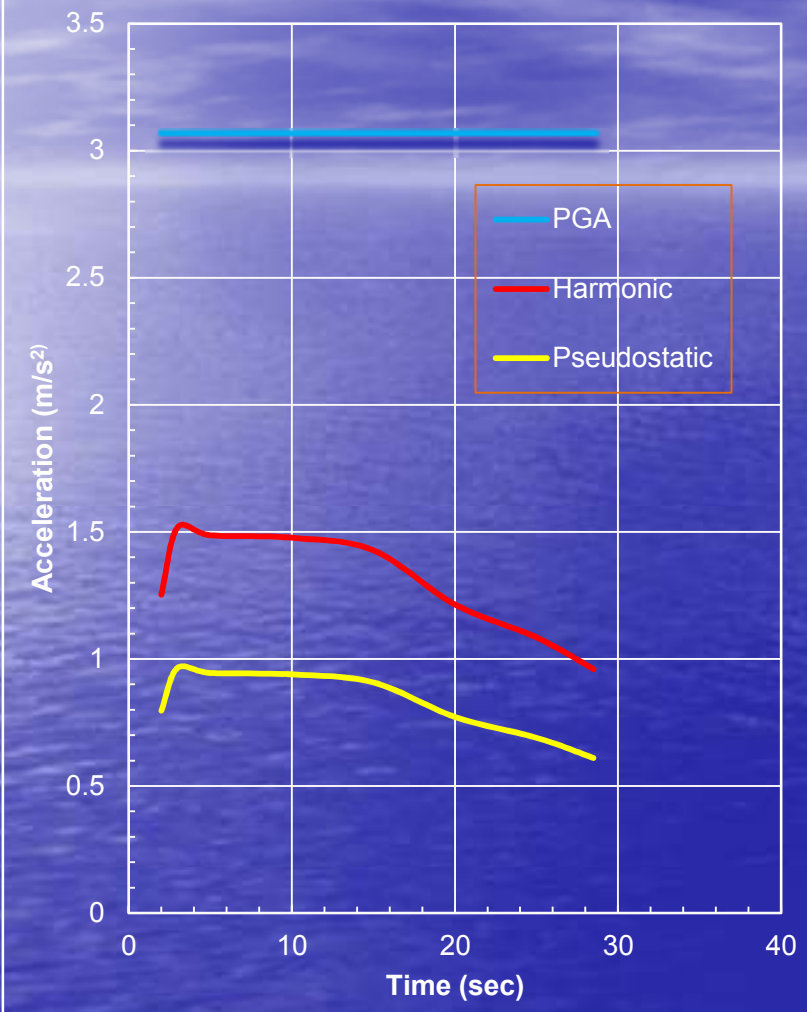
- Velocity time history

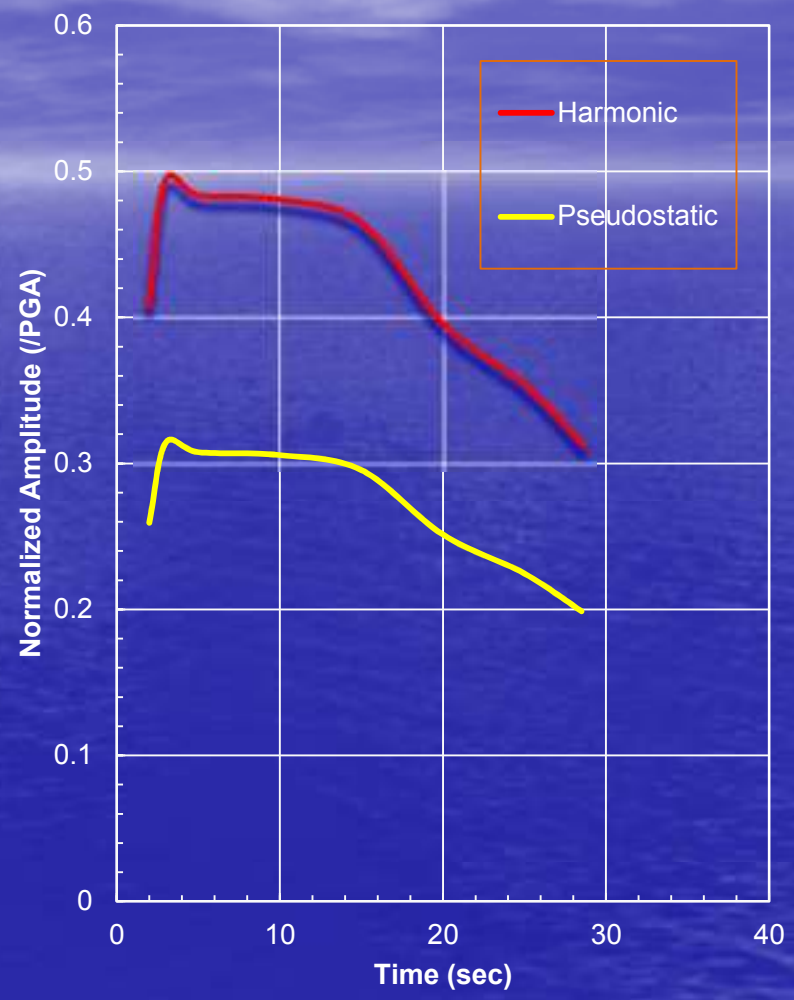


- Displacement time history



Equivalent cyclic wave Amp & Pseudostatic k_H





Kobe Earthquake (1995), Type111

Suggested k_H – PGA relationship

- Essential Structures

$$k_H = \frac{1}{2}\,PGA$$

- Other Structures

$$k_H = \frac{1}{3}\,PGA$$

STATIC DESIGN VS. SEISMIC DESIGN

Static Design vs. Seismic Design

- Factor of safety for static design

$$FS_{static} = \frac{S}{F_{static}} = 1.5$$

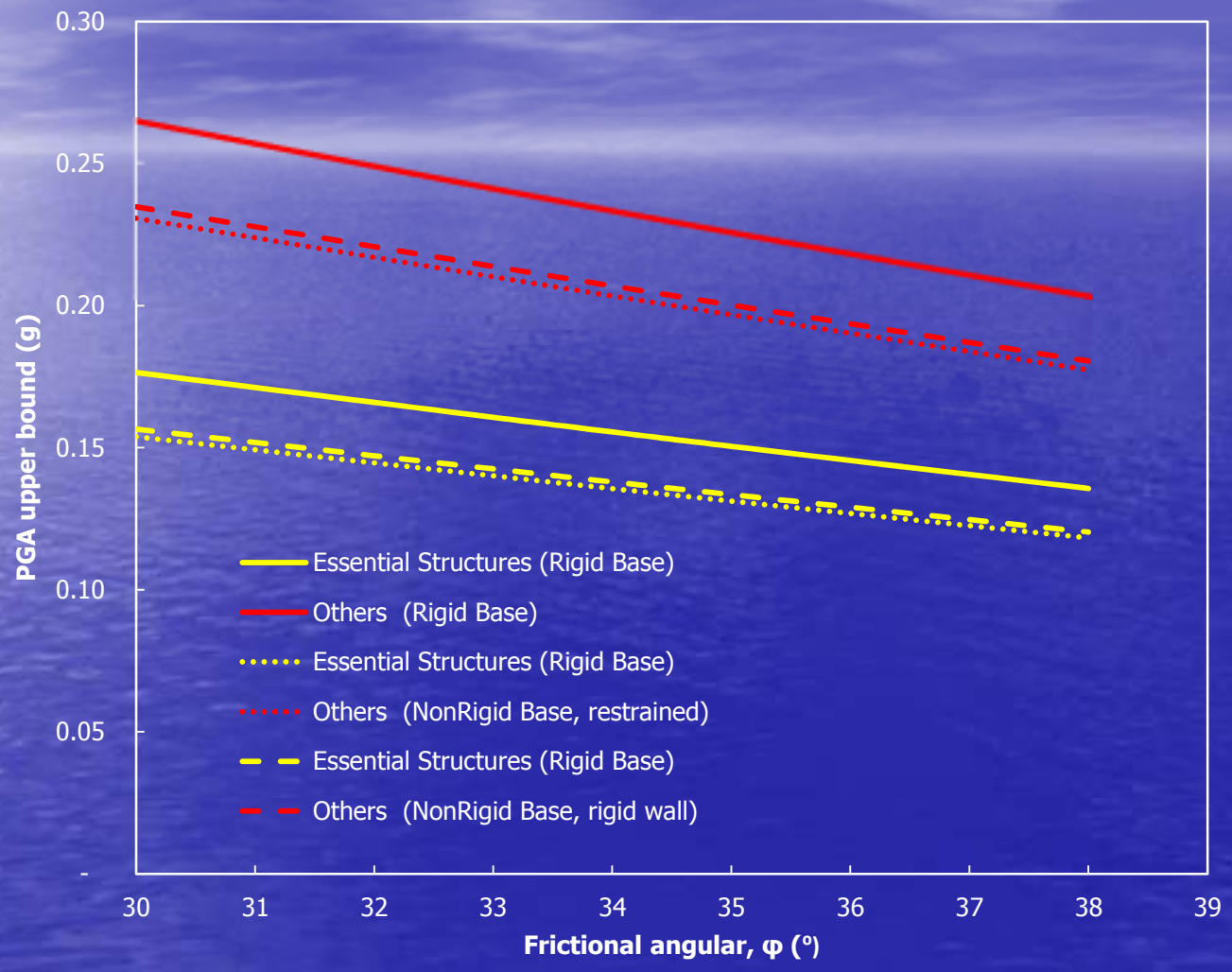
- Factor of safety for seismic design

$$FS_{seismic} = \frac{S}{F_{seismic}} = 1.1$$

- Therefore, a statically designed structure can withstand a seismic load of

$$F_{seismic} = \frac{1.5}{1.1} F_{static} = 1.36 F_{static}$$

Upper Bound of PGA covered by static design



CONCLUSIONS AND RECOMMENDATIONS

Conclusions

- The increment of SEP of rigid walls is independent of internal frictional angle of the backfill soils
- The ratio of the increment of SEP of rigid walls to γH^2 depends only on k_h and base conditions
- The increment can be expressed as
  - $\Delta P_{0E} = \alpha_k \cdot k_H \cdot \gamma H^2$
  - $\alpha_k = 1.02\ for\ rigid\ base\ \&\ \alpha_k = 1.17\ for\ non-rigid\ base$
- Thrust point is lower than the middle point of the wall and can be conservatively taken as 0.45H
- K_h is significantly lower than PGA and can be conservatively taken as
  - $k_H = \beta \cdot (PGA)$
  - $\beta = {}^1/_2\ for\ essential\ structures$
  - $\beta = {}^1/_3\ for\ other\ structures$

Recommendations

- Seismic Earth Pressures under Restrained Condition

$$\Delta P_{0E} = \alpha \cdot (PGA) \cdot \gamma H^2$$

Type of Structures	α	
	Rigid Base	Non-Rigid Base
Essential Structures	0.5	0.6
Other Structures	0.33	0.4

- Thrust point=0.45H

THANK YOU!