Evaluating the Seismic Coefficient for Slope Stability Analyses

by

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AGENDA

What is the Seismic Coefficient, k_s?

Factors affecting k_s

Historical approach

Modern approach

Example

Summary / Conclusions

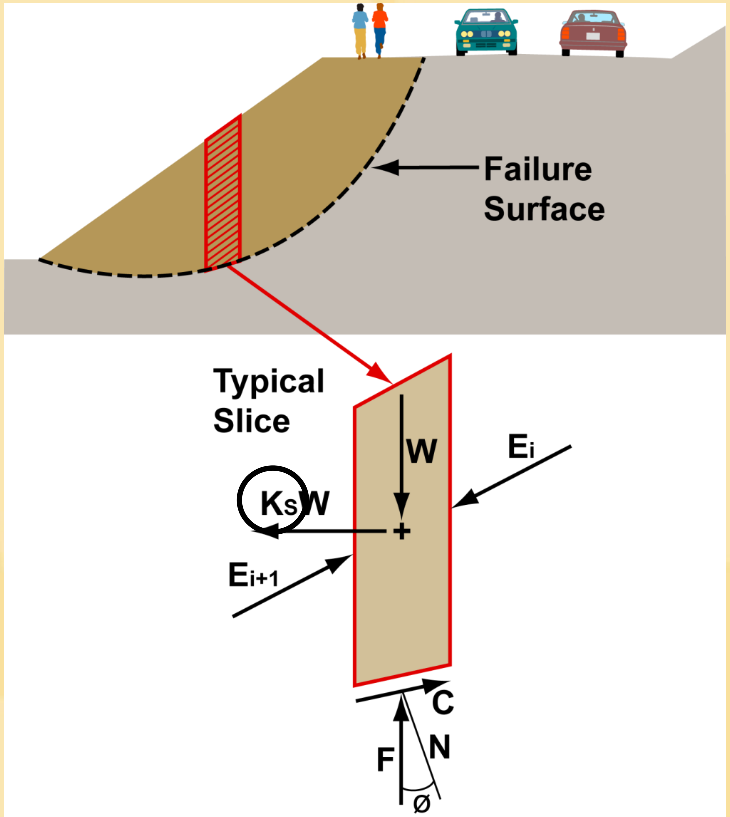
Questions / Answers

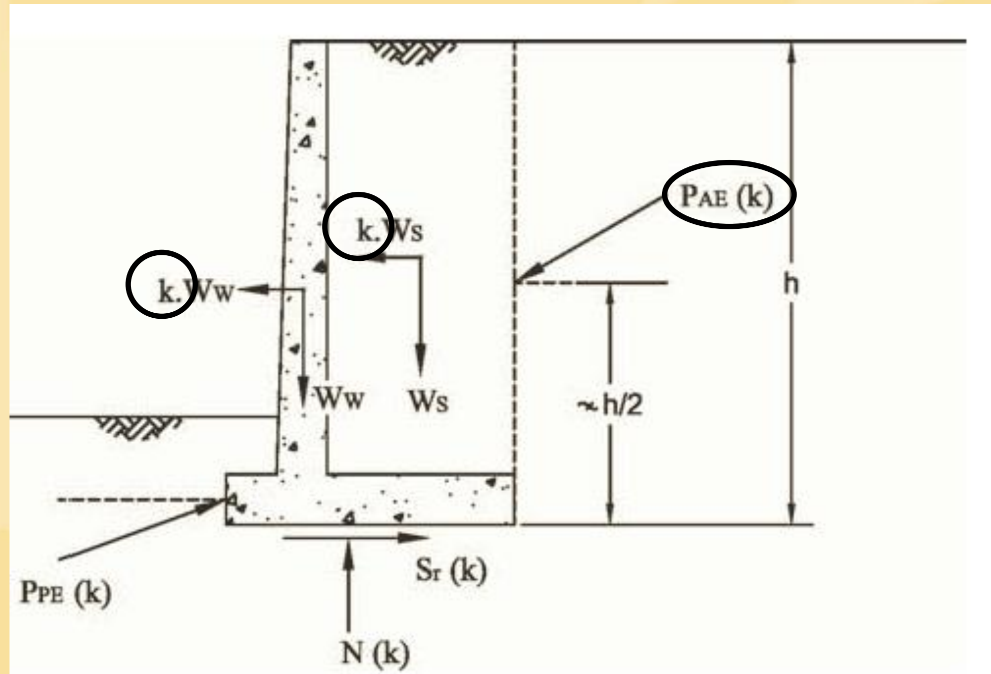
What is the seismic coefficient, k_s?

The seismic coefficient is:

- A lateral force coefficient used in pseudo static limit equilibrium analysis
- A means of representing the effect of seismic loading on slopes and earth retaining structures using limit equilibrium analysis

What is the seismic coefficient, k_s?





What the seismic coefficient **is not**.

The seismic coefficient **is not**:

- The same as the peak ground acceleration (PGA) [not usually]
- A vertical force coefficient
- Independent of the factor of safety

PGA vs. k_s

PGA (peak **horizontal** ground acceleration) occurs at one point

- Acceleration elsewhere is less than PGA
- PGA may only occur one time during the EQ

k_s is an average value over entire mass

➡ k_s is usually less than (and never more than) the PGA (÷ g)

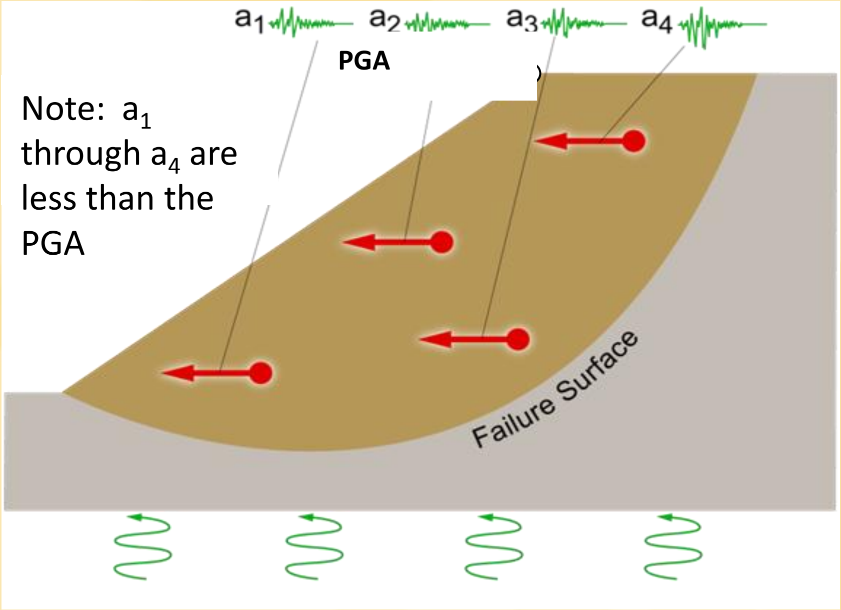
Note: k_s = PGA / g for brittle and/or sensitive soil (due to progressive failure)

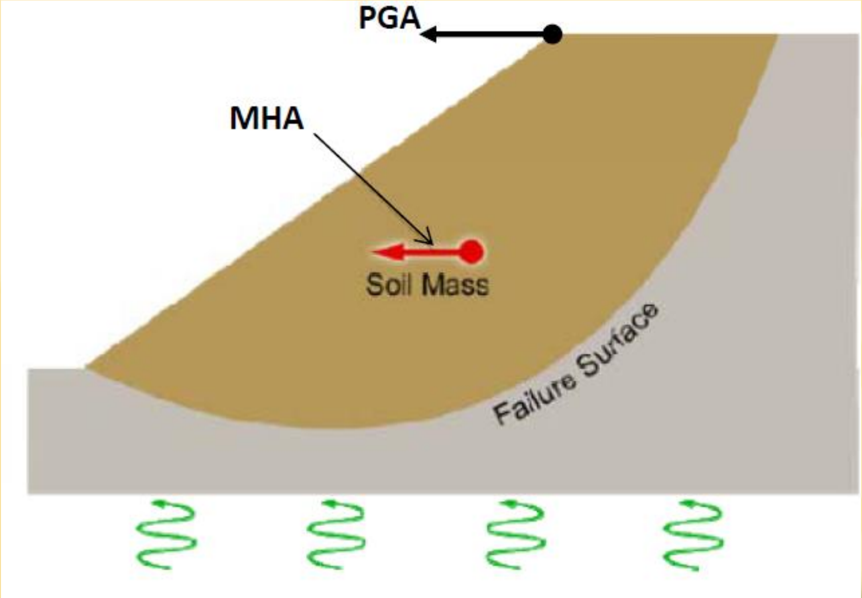
Maximum Horizontal Acceleration

MHA

- Maximum Average Horizontal Acceleration of failure mass
- Governs maximum horizontal inertial force on failure mass

PGA ≥ MHA (so PGA ≥ MHA ≥ k_s)





Factors Influencing k_s

The value of k_s may depend upon:

- The associated factor of safety
- The seismic performance criteria
- The design ground motions
- Slope height

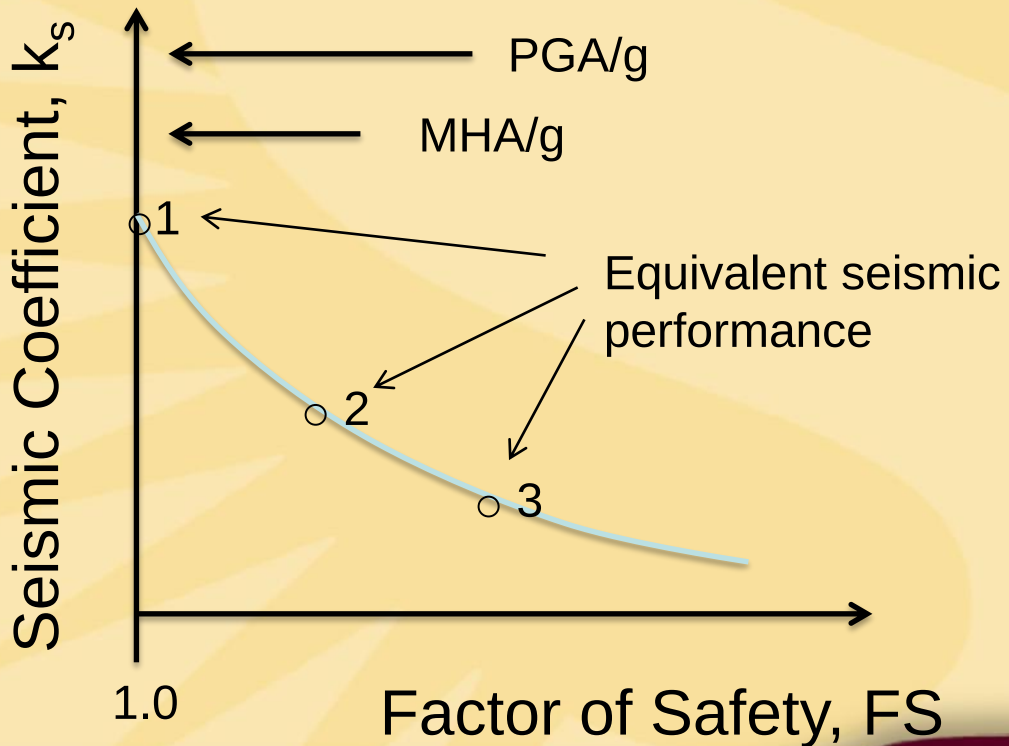
k_s – FS Coupling

- Specifying k_s without an associated FS is meaningless
- Specifying a “seismic FS” without specifying an associated k_s is meaningless
- Different combinations of k_s and FS can describe an equivalent performance standard
  - Increase FS, decrease k_s

Seismic Factor of Safety

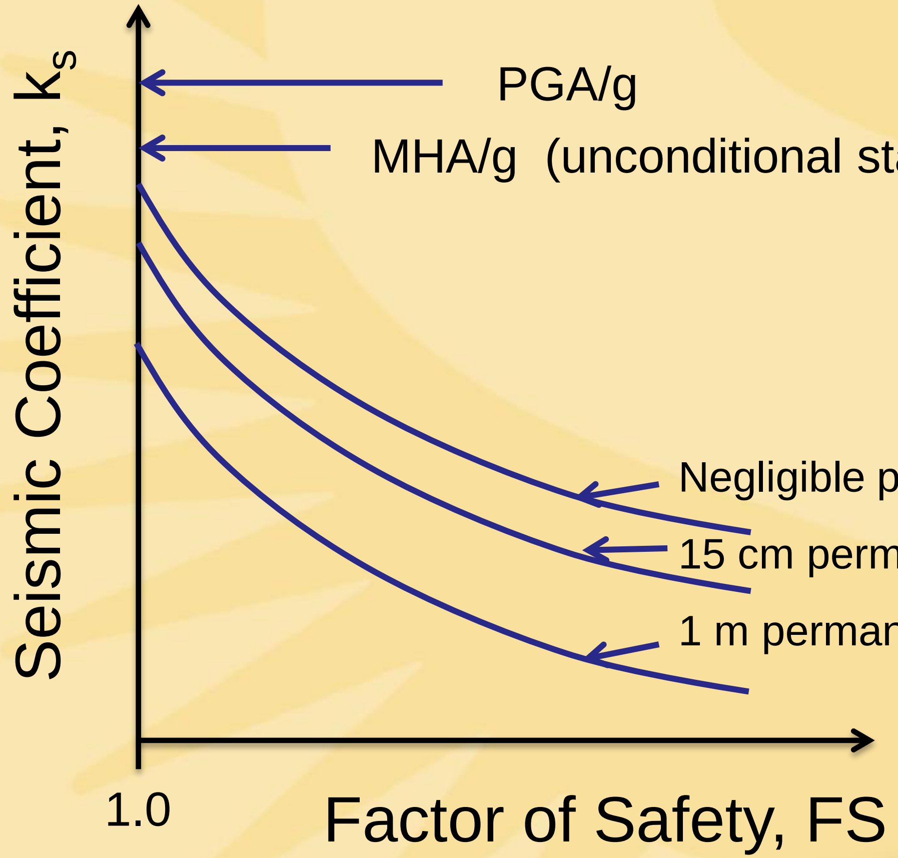
Specifying k_s without an associated FS is meaningless

$$[(k_s)_1, FS_1] \equiv [(k_s)_2, FS_2] \equiv [(k_s)_3, FS_3]$$



Seismic Performance Criteria

Different performance criteria correspond to different k_s, FS combinations



Seismic Performance Criteria

- Unconditional seismic stability is elusive
  - May not be obtainable in high seismicity areas
  - Probably not necessary
- Seismic performance usually quantified by allowable permanent displacement
  - Negligible (minor cracking)
  - Small (inches)
  - Large (feet)
  - Instability (tens of feet)

Seismic Performance Criteria

MHA – k_s relationship depends upon seismic performance criteria

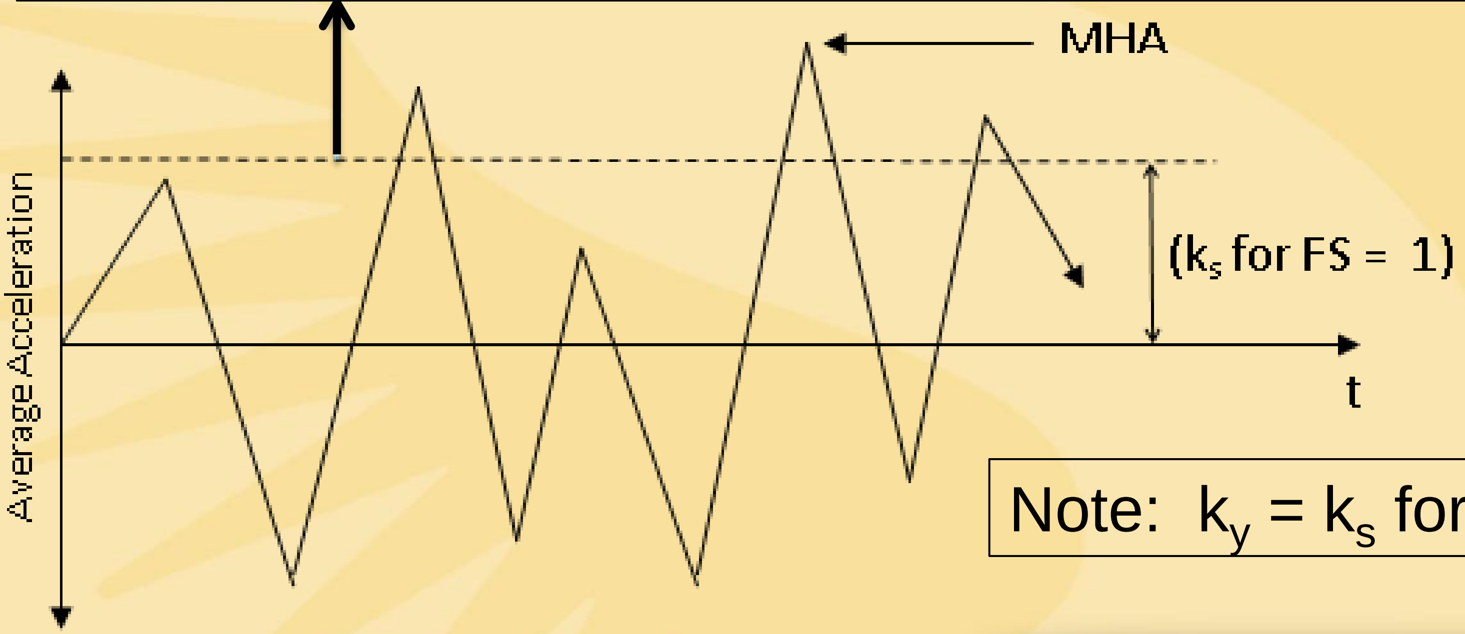
- Unconditional stability: k_s = MHA, FS = 1
  - Exception: Soils susceptible to progressive failure (use k_s = PGA)
- Allowable displacement: k_s < MHA
  - Increase allowable displacement, decrease k_s

Allowable Displacement

Allowable displacement = f(soil ductility, impacts of slope displacement)

Greater allowable displacement, smaller k_s

Permanent displacement accumulates when average acceleration > k_y



Note: $k_y = k_s$ for FS = 1

Design Ground Motions

Design ground motions influence:

- The relationship of the PGA to the MHA
  - Factors include ground motion frequency, slope height
- The relationship of the MHA to k_s
  - Factors include performance criteria (allowable displacement), frequency and duration of motion

Relationship of PGA to MHA

PGA – MHA relationship impacted by spatial and temporal incoherence (variability)

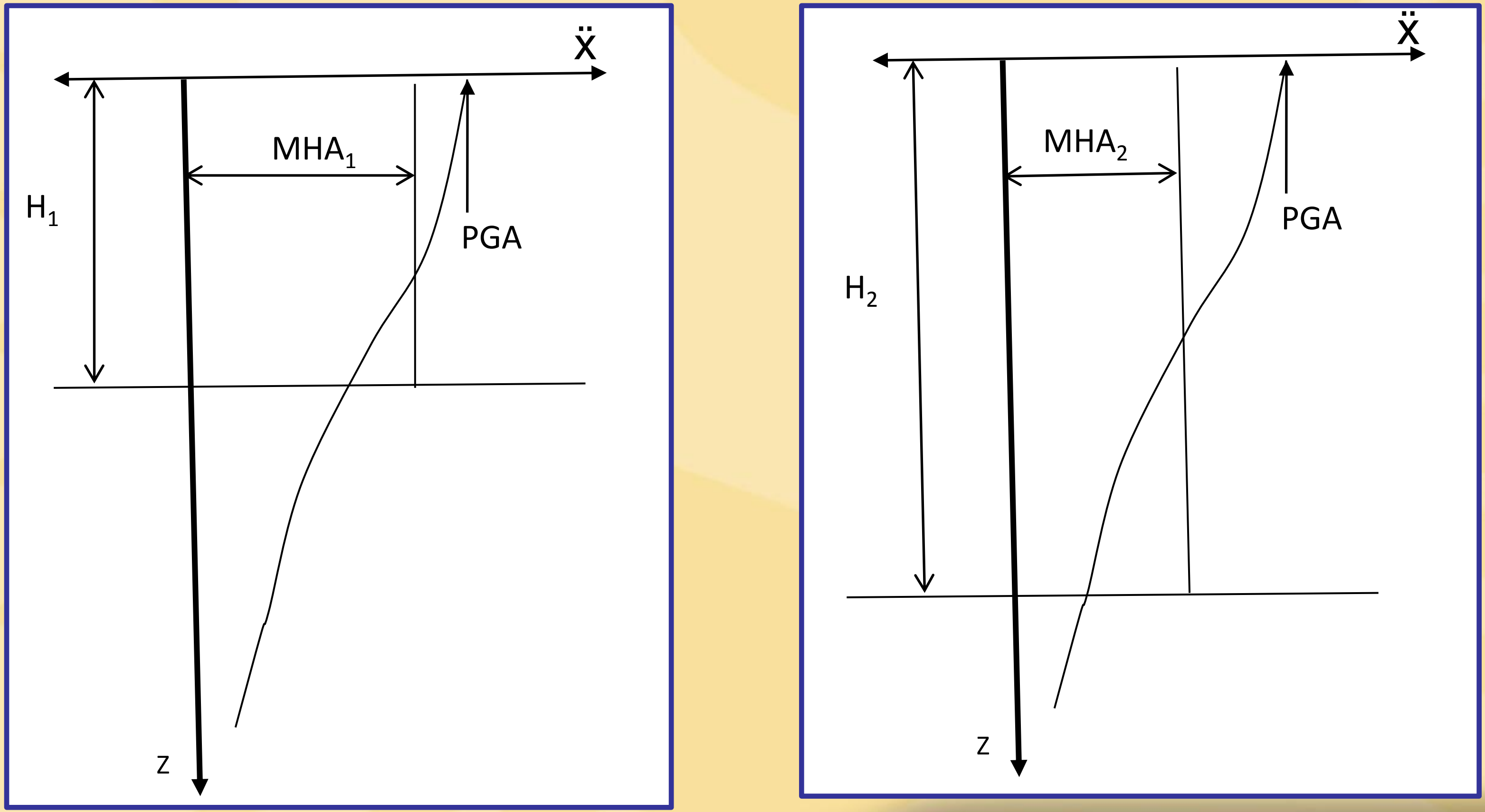
- Maximum acceleration at all other points is less than PGA
- Maximum acceleration at other points occurs at different time than PGA

➡ Maximum average acceleration (MHA) is less than PGA

Influence of Slope Height

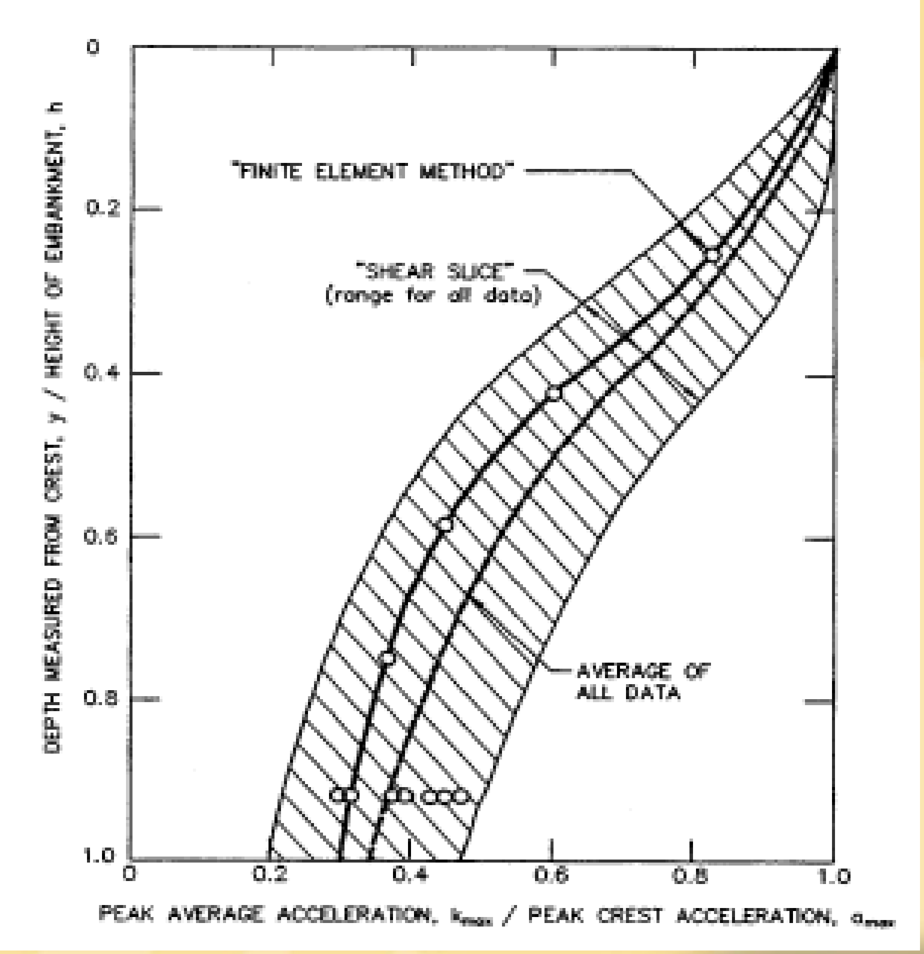
Increasing H reduces MHA (more averaging)

- $H_1 < H_2$, then $MHA_1 > MHA_2$



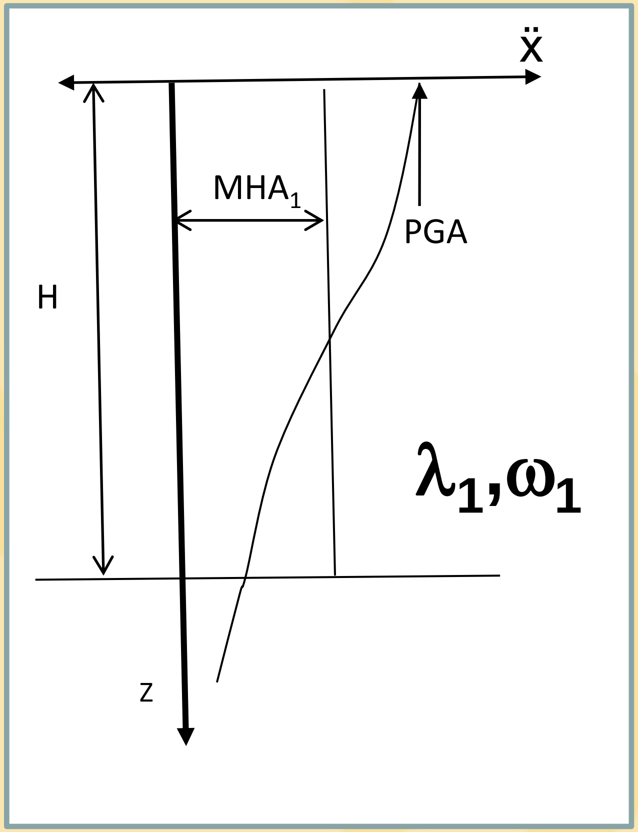
Influence of Slope Height

Makdisi and Seed (1978)



Influence of Frequency on MHA

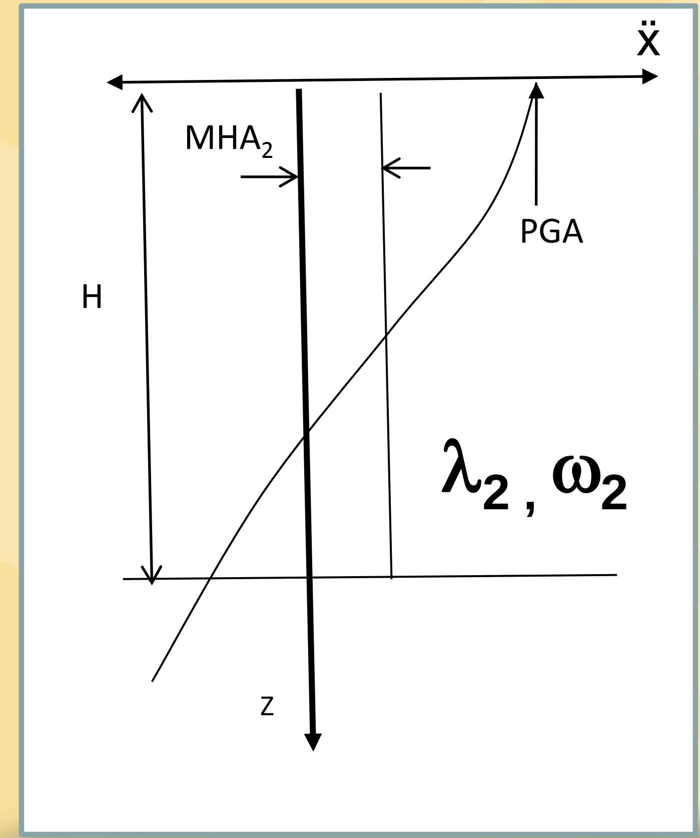
Higher frequency (ω), shorter wave length (λ), reduced MHA



$\omega_1 < \omega_2$

$\lambda_1 > \lambda_2$

$MHA_1 > MHA_2$



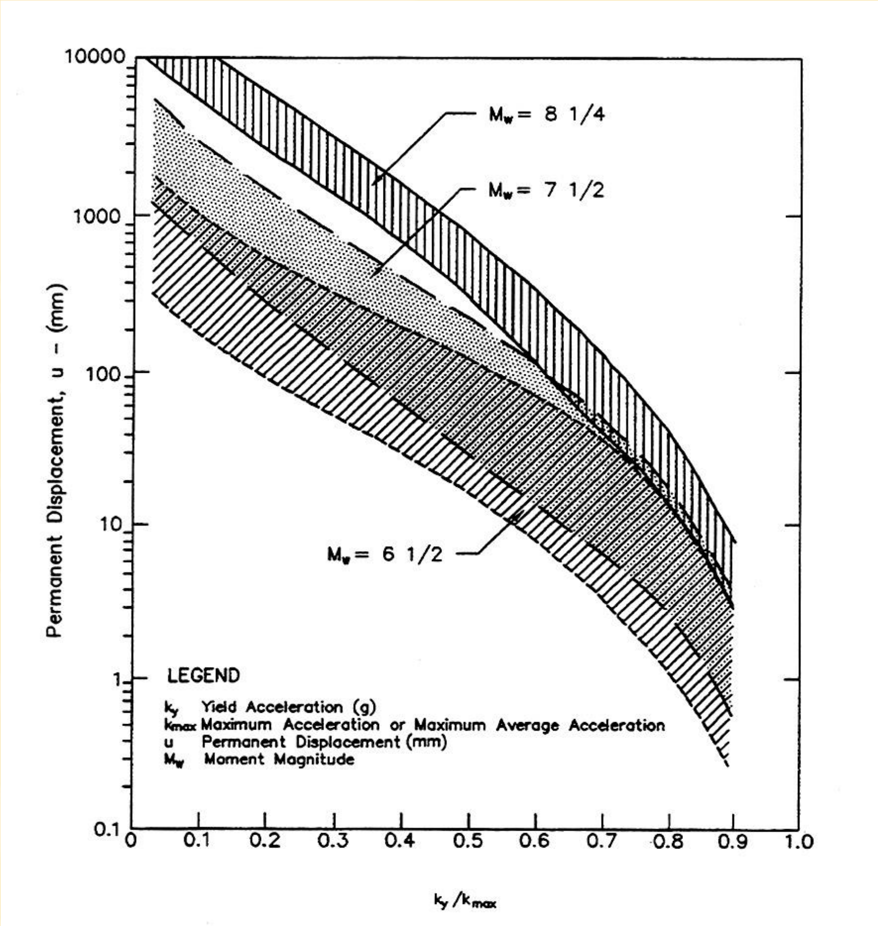
Influence of Duration, Frequency

- Increased duration, larger displacement potential, smaller reduction in k_s from MHA
- Higher frequency, more cycles of loading, but shorter cycles – impact unclear

Both duration and frequency effects on MHA-k_s relationship traditionally captured as magnitude dependence

Influence of Magnitude

Makdisi and Seed, 1978

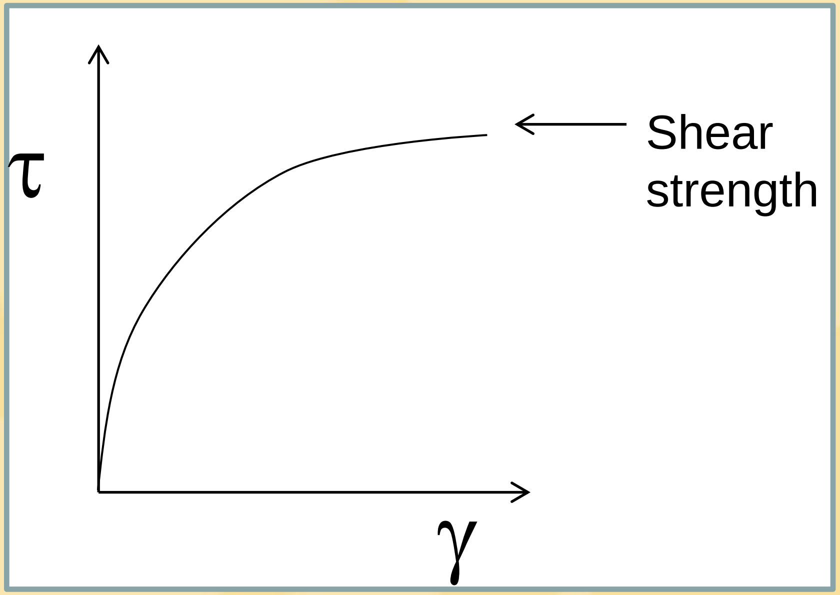


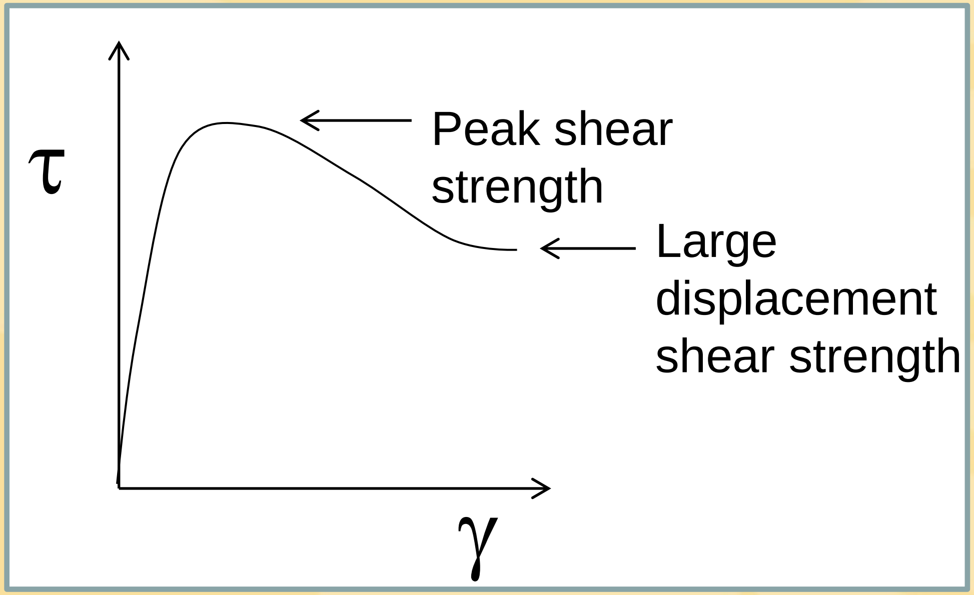
Other Factors Influencing k_s

- Shear strength
  - Peak vs. large displacement
  - Cyclic softening
- Multiple failure surfaces
- Amplification of ground motions
  - Rock vs. soil site motions
  - Influence of topography

Peak vs. Large Displacement Shear Strength

In a non-ductile soil, use large displacement shear strength (by convention / conservative)





Cyclic Softening

- Reduce soft clay shear strength for cyclic softening
  - Typically reduce S_u by 10-20%
- Use residual shear strength in liquefiable soil

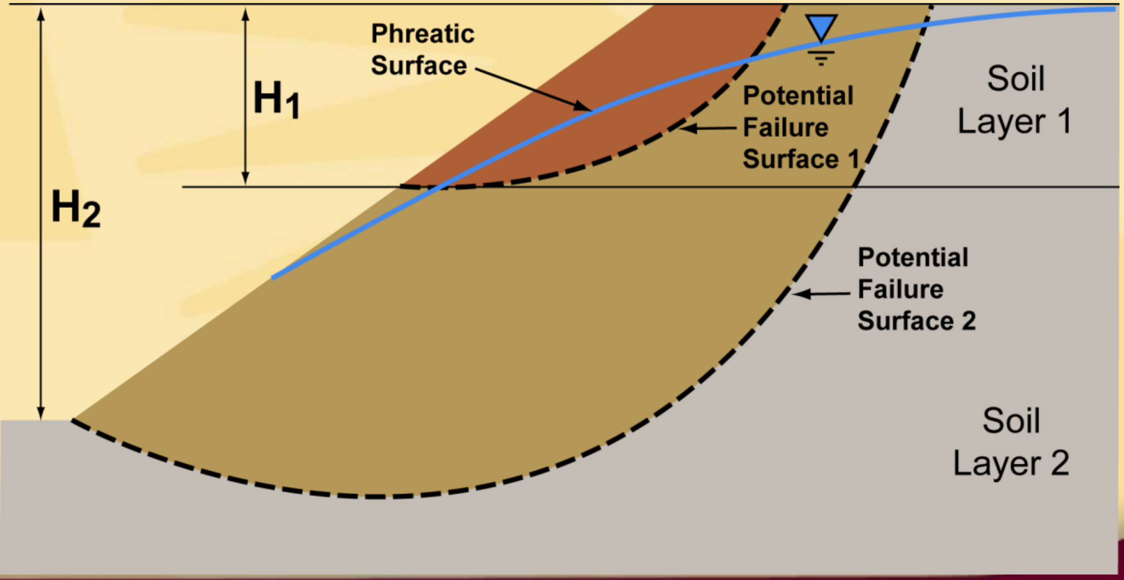
Multiple Failure Surfaces

Note that α is a function of H

- MHA decreases with depth

Stability also may decreases with depth

- May need to check multiple surfaces
- Ratio of k_y (k_s for FS = 1) to MHA critical



$$\alpha_{H1} > \alpha_{H2}$$

$$(MHA)_{H1} > (MHA)_{H2}$$

$$(k_y)_1 > (k_y)_2$$

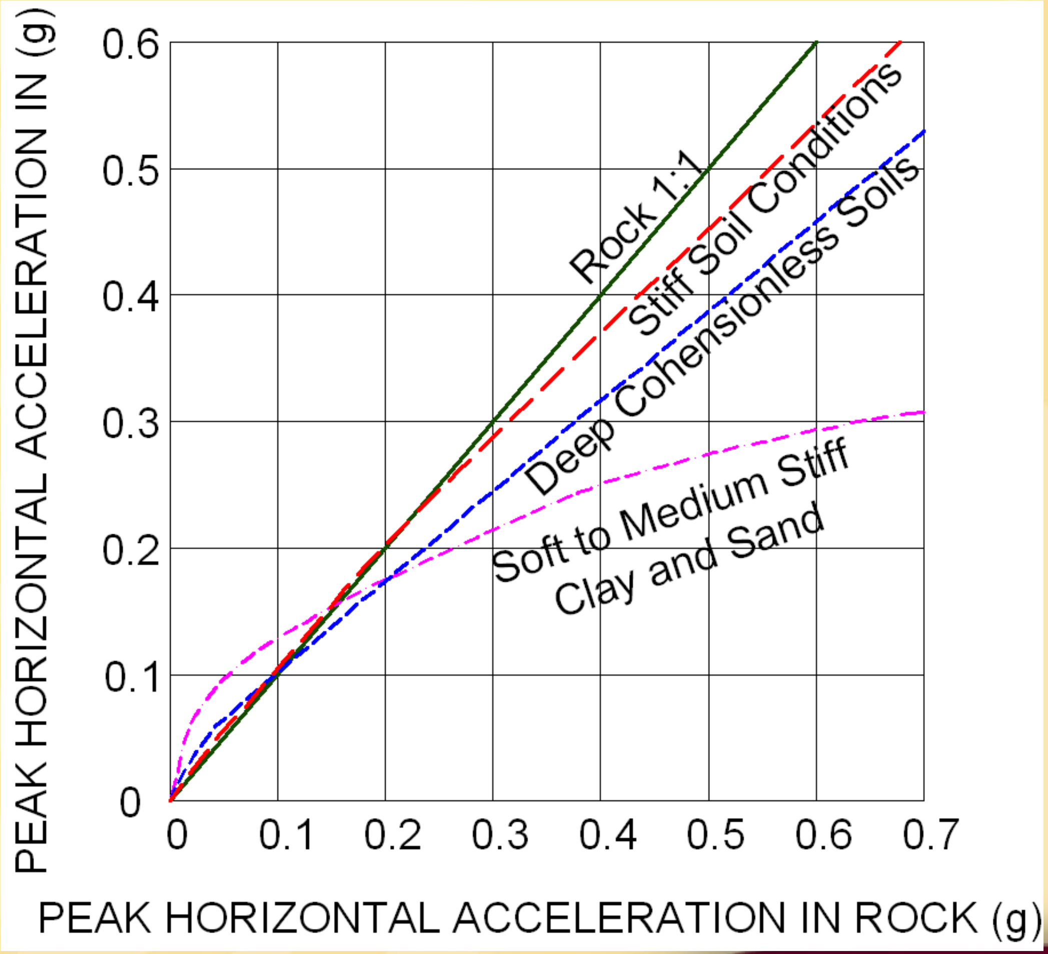
$$(k_y/MHA)_1 \text{ vs. } (k_y/MHA)_2 \text{ ???}$$

Ground Motion Amplification

- Seismic hazard maps typically developed for a reference site condition
  - US: Site Class B (“B/C boundary per USGS)
  - Canada: Site Class C
- Ground motions (PGA and S_a) must be adjusted for other site conditions
  - Code values adjusted using site factors
- Can also have topographic amplification

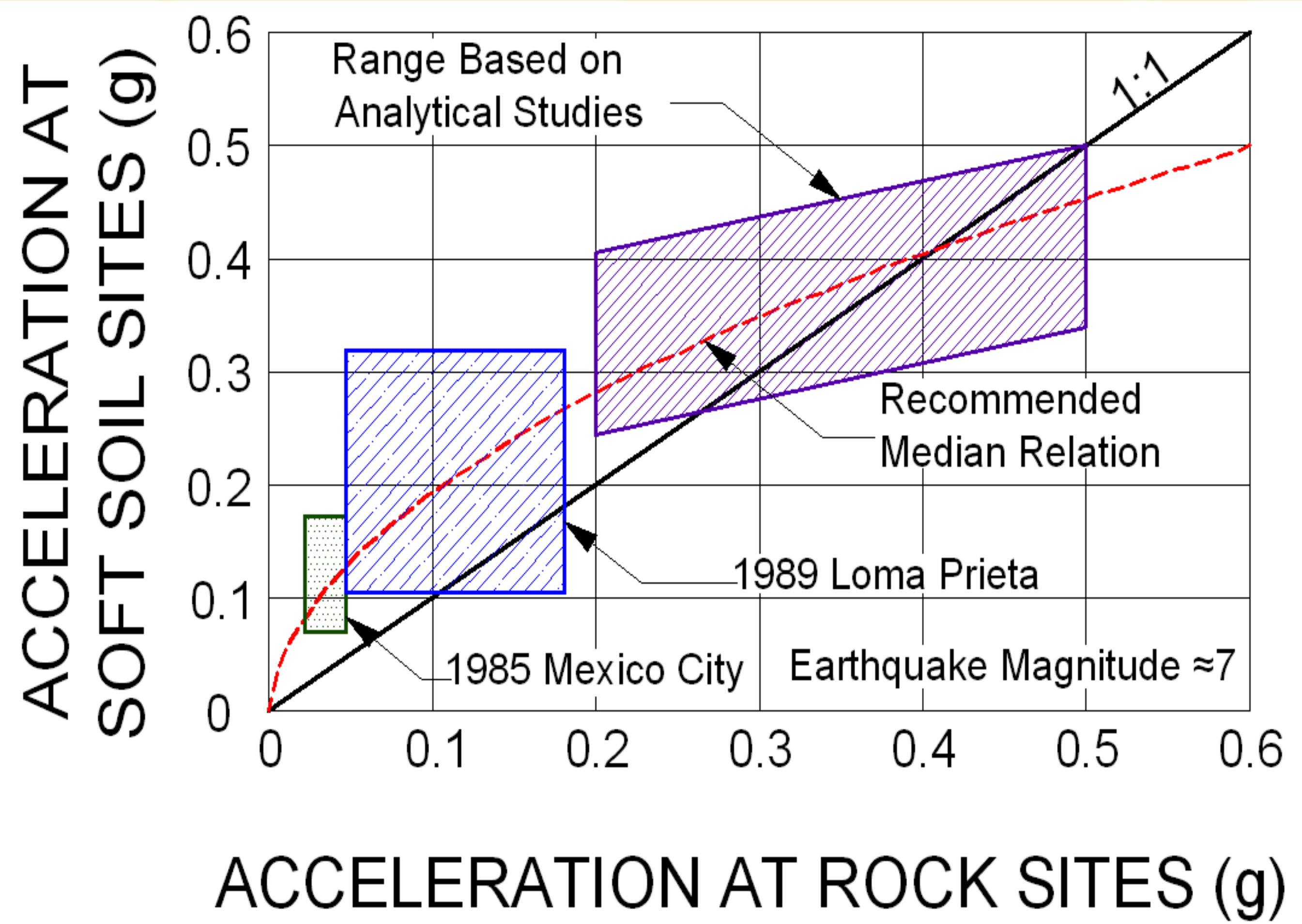
PGA Amplification

Seed and Idriss, 1982: Rock vs. Soil Sites



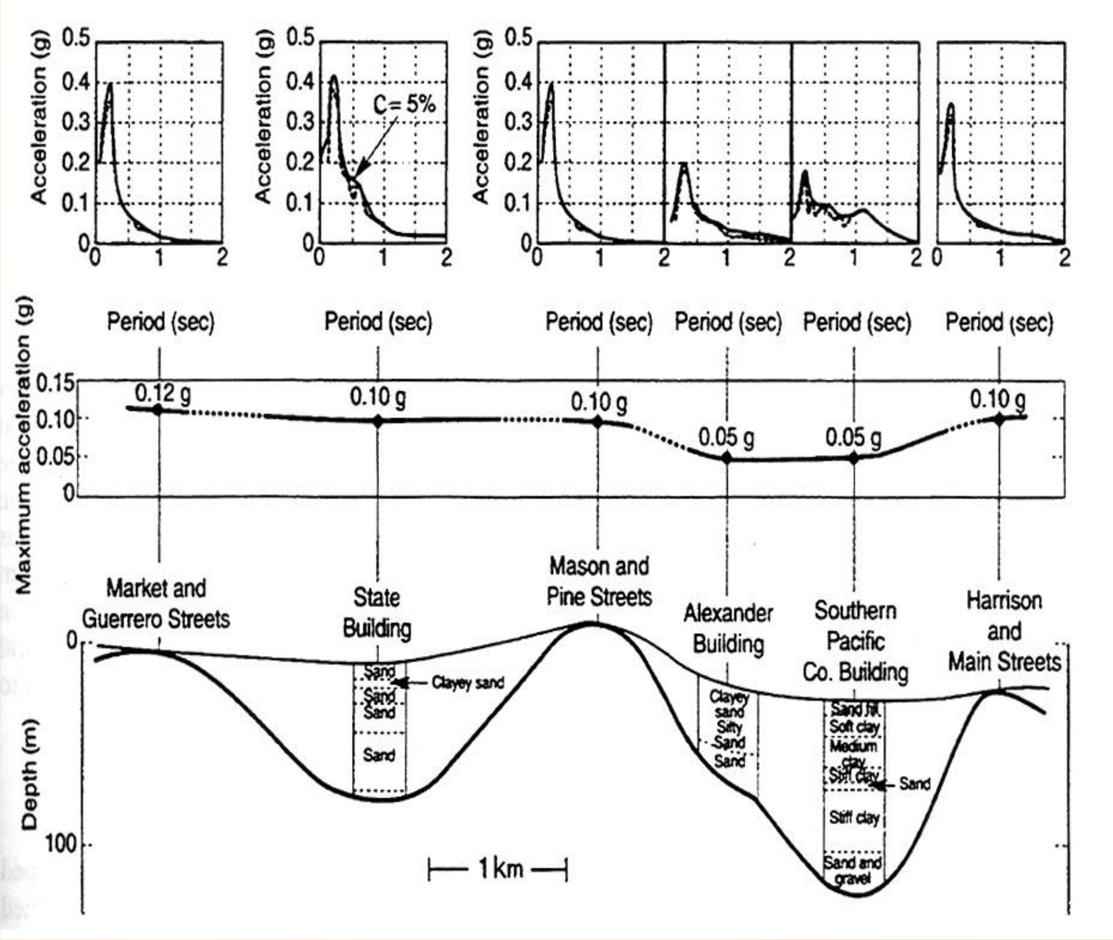
PGA Amplification

Idriss, 1992: Rock vs. Soft Clay Sites



Spectral Amplification

1957 Daly City Earthquake (Seed, 1975)



Site Class

Based on average shear wave velocity in top 100 ft (30 m), $(V_S)_{30}$ (or other geotech characteristics)

Site Class	**$(V_s)_{30}$**	**SPT**	**S_u**
A	> 5000 ft/s	N.A.	N.A.
B	2500 - 5000 ft/s	N.A.	N.A.
C	1200 - 2500 ft/s	> 50	> 2 ksf
D	600 - 1200 ft/s	15 - 50	1 -2 ksf
E	< 600 ft/s	<15	< 1 ksf
F	(Special Study Sites)		

PGA Site Factor, FPG_A (NBCC Values)

$$PGA = PGA_{\text{Site Class C}} \times F_{PGA}$$

Site Class	Peak Ground Acceleration for Site Class C				
	PGA ≤ 0.10 g	PGA = 0.20 g	PGA = 0.30 g	PGA = 0.40 g	PGA ≥ 0.50 g
A	0.7	0.7	0.8	0.8	0.8
B	0.8	0.8	0.9	1.0	1.0
C	1.0	1.0	1.0	1.0	1.0
D	1.3	1.2	1.1	1.1	1.0
E	2.1	1.4	1.1	0.9	0.9
F	*a*	*a*	*a*	*a*	*a*

Table notes: Use straight line interpolation for intermediate values of *PGA*, where PGA is the peak ground acceleration obtained from the ground motion maps.

a Site-specific geotechnical investigation and dynamic site response analyses shall be performed

Long Period Site Factor, F_V (NBCC)

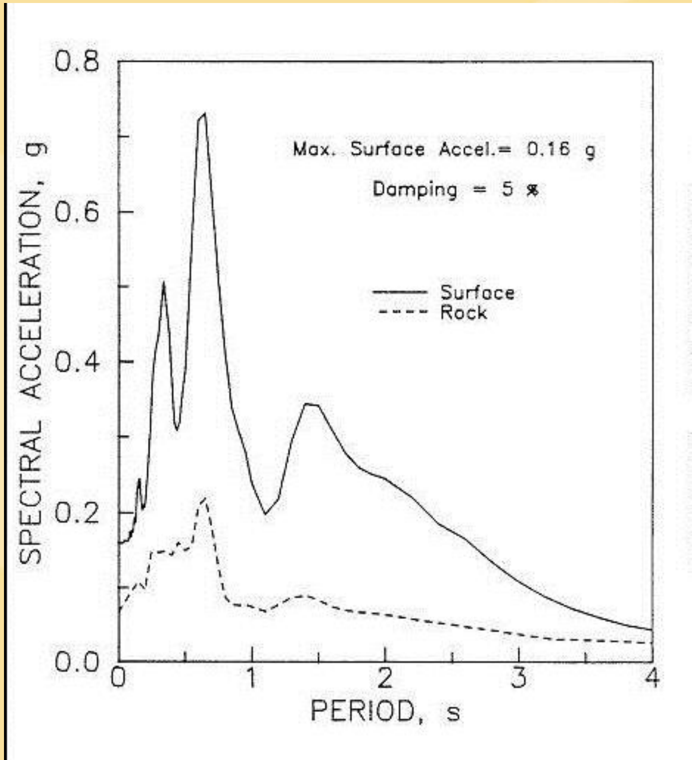
$$S_1 = (S_1)_{\text{Site Class C}} \times F_V$$

Site Class	Spectral Acceleration at 1 Sec Period, S_1. for Site Class B				
	$S_1 \le$ 0.10 g	$S_1 =$ 0.20 g	$S_1 =$ 0.30 g	$S_1 =$ 0.40 g	$S_1 \ge$ 0.50 g
A	0.5	0.5	0.5	0.6	0.6
B	0.6	0.7	0.7	0.8	0.8
C	1.0	1.0	1.0	1.0	1.0
D	1.4	1.3	1.2	1.1	1.1
E	2.1	2.0	1.9	1.7	1.7
F	*a*	*a*	*a*	*a*	*a*

Table notes:	Use straight line interpolation for intermediate values of S_1, where S_1 is the spectral acceleration at 1.0 seconds obtained from the ground motion maps.
a	Site-specific geotechnical investigation and dynamic site response analyses shall be performed

Special Study Sites

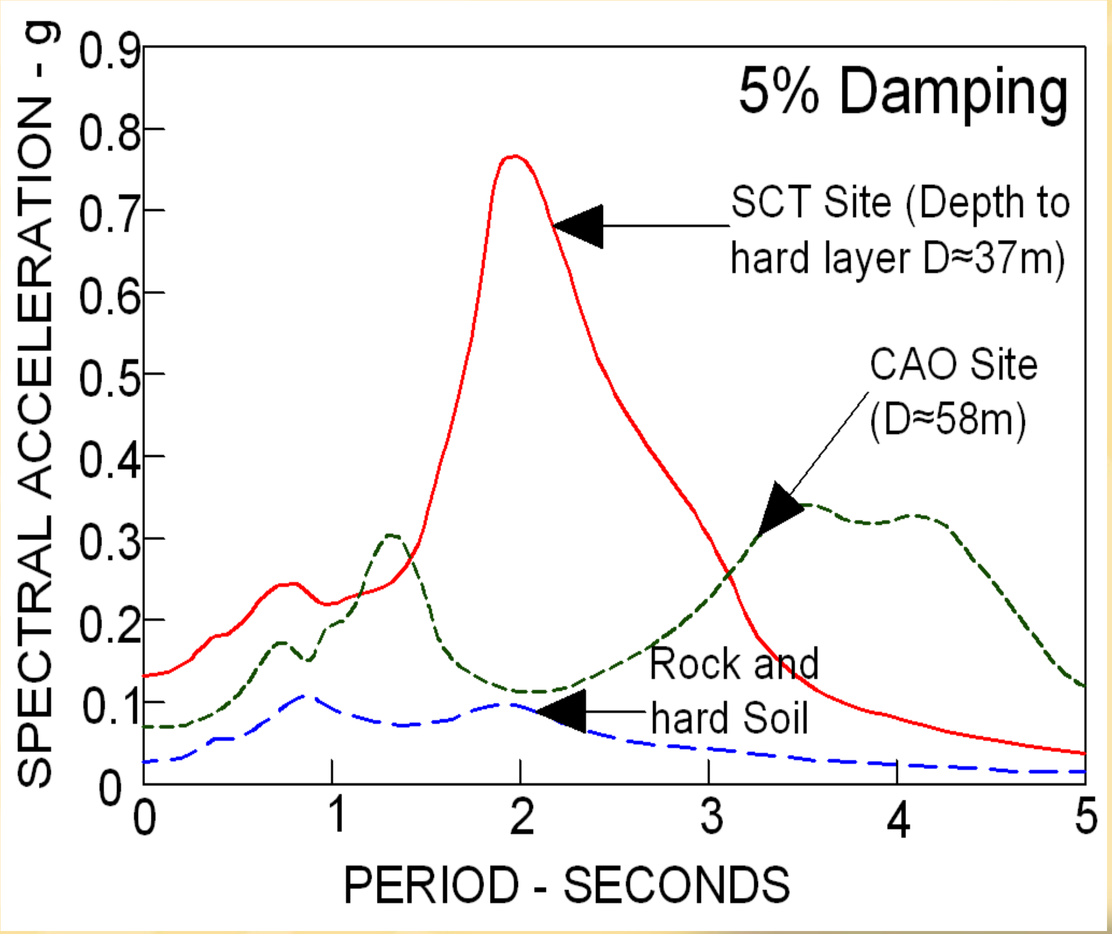
Yerba Buena Island (Rock) / Treasure Island (Soil) sites in the 1989 Loma Prieta Earthquake



5.6(a): 90 Degree (E-W) Component

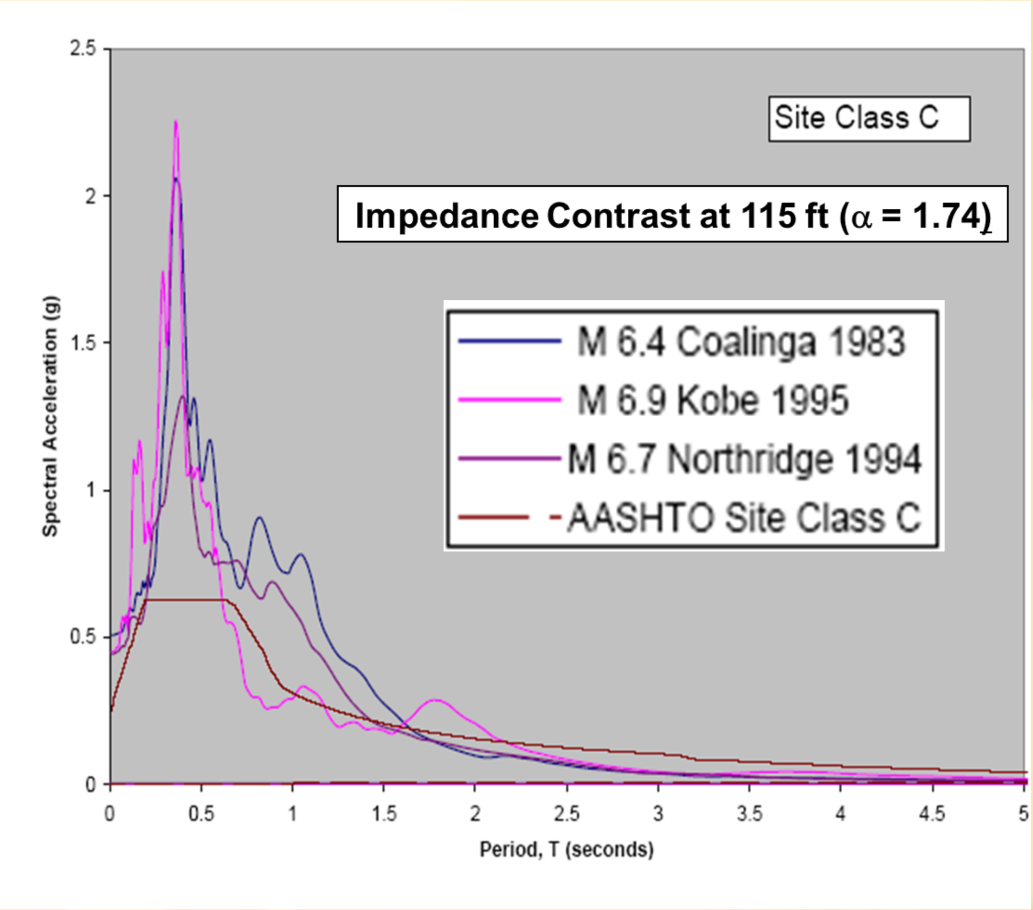
Special Study Sites

Mexico City, 1985



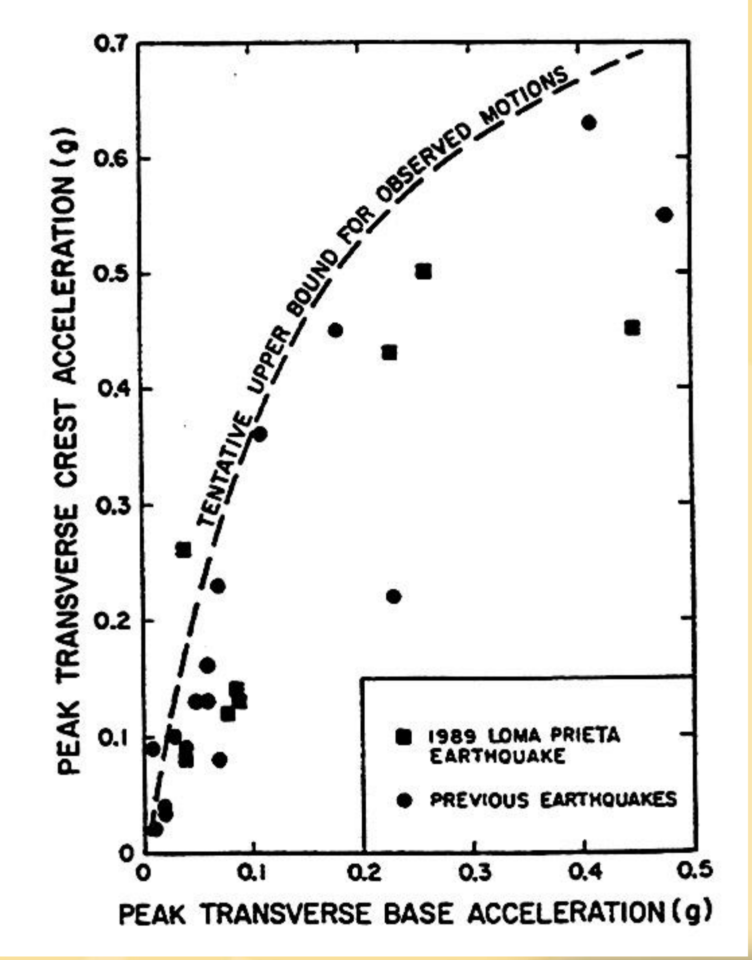
Special Study Sites

Shallow Stiff Layer Sites



Topographic Amplification

Harder, 1991: Embankments Response



Historical k_s Values

Seed, 1979: For "acceptable" displacement of earthen dams (displacement ≤ 1 m)

- k_s = 0.15, FS = 1.15 for PGA ≤ 0.75g, M ≤ 7.5
- k_s = 0.10, FS = 1.15 for PGA ≤ 0.75g, M ≤ 6.5

Notes:

- Both k_s and FS specified
- Influence of earthquake magnitude on k_s
- k_s/PGA = 0.167 for M 7.5, k_s/PGA = 0.133 for M 6.5
- Not valid for liquefiable soil, 15% strength reduction for soft clay

Historical Values

Hynes and Franklin, 1984: Also for 1 m acceptable displacement of earth dams

- $k_s = 0.5\ PGA_{(free\ field)}$ for $M \leq 8.3$

Notes:

- No consideration of magnitude dependence
- Assumes PGA amplification of 3 from base to top of embankment ($k_s/PGA_{crest} = 0.167$)
- Reduce soft clay strength by 20% for cyclic softening

Historical Values

FHWA (1997): For acceptable performance of slopes and retaining structures for transportation facilities:

- $k_s/PGA_{(free\ field)} = 0.5$
- Acceptable performance = 15 cm (6 in.) displacement

Historical Values

Kavazanjian, 1998:
Values of k_s/PGA as a function of allowable displacement (based upon Hynes and Franklin):

Displacement	M ≤ 6.5 and D ≥ 10 km	M > 6.5 or D < 10 km
100 mm	0.23	0.35
150 mm	0.17	0.27
300 mm	0.08	0.17
500 mm	0.05	0.11
1 m	0.03	0.06

Note:

- PGA includes amplification effects

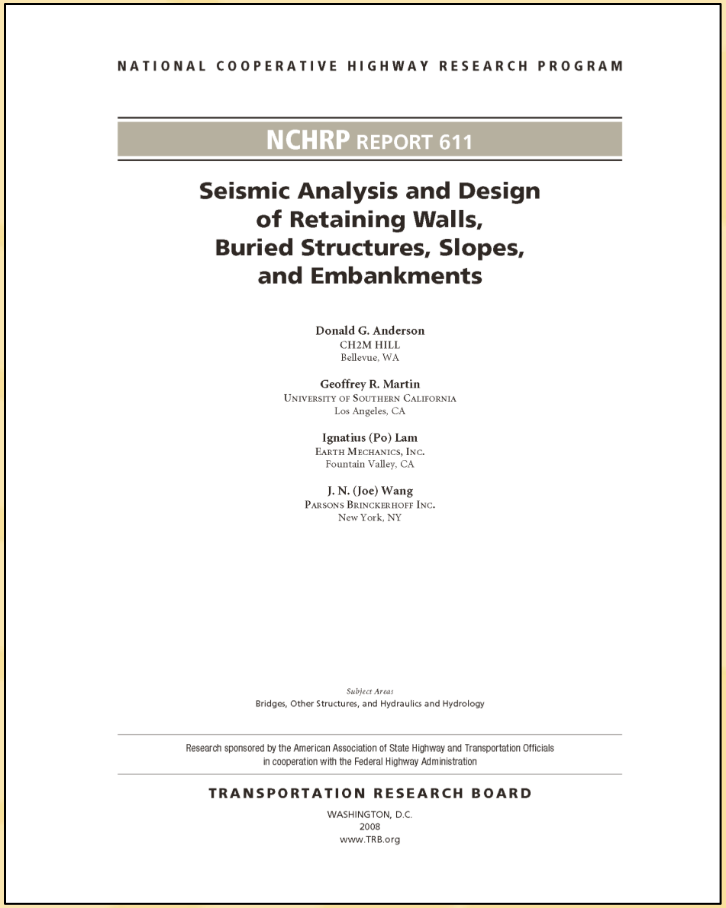
Modern Approaches

Displacement-based values for the seismic coefficient (and factor of safety)

- NCHRP 12-70 / FHWA 2011
- Bray and Travasarou, 2009

Note: Both methods can be applied to slopes and walls.

NCHRP 12-70 / FHWA (2011)



NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP REPORT 611

Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments

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Subject Areas
Bridges, Other Structures, and Hydraulics and Hydrology

Research sponsored by the American Association of State Highway and Transportation Officials in cooperation with the Federal Highway Administration

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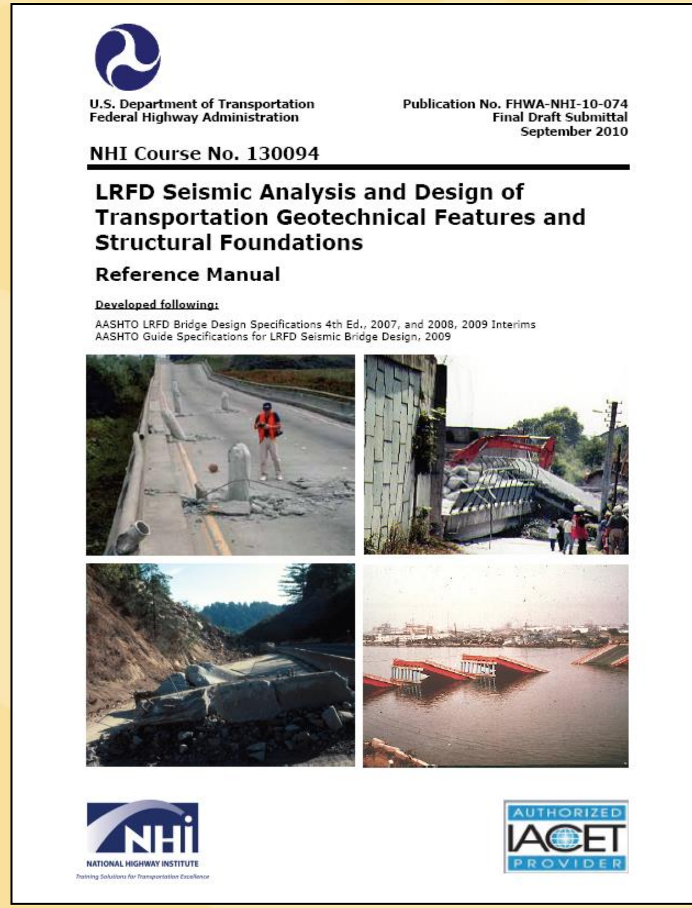


U.S. Department of Transportation
Federal Highway Administration

Publication No. FHWA-NHI-10-074
Final Draft Submittal
September 2010

NHI Course No. 130094

LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations

Reference Manual

Developed following:

AASHTO LRFD Bridge Design Specifications 4th Ed., 2007, and 2008, 2009 Interims
AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2009

http://www.trb.org/Main/Blurbs/Seismic_Analysis_and_Design_of_Retaining_Walls_Bur_160387.aspx
http://www.fhwa.dot.gov/engineering/geotech/library_arc.cfm?pub_number=19

NCHRP 12-70 / FHWA (2011)

Based upon:

- Finite element analysis to get MHA, average acceleration time history
- Newmark analysis to get seismic displacement from average acceleration time history
- Sensitivity study to establish k_s as a function of permanent seismic displacement, FS

NCHRP 12-70 / FHWA (2011)

k_s = MHA x r

- r = soil ductility factor [r ≤ 1]

MHA = α x PGA

- PGA is site-specific value
- $\alpha = f(H, \beta)$ [a ≤ 1]
  - H = slope height
  - β captures frequency, duration (i.e. magnitude) effects

FS = f(performance criteria)

- Allowable displacement

NCHRP 12-70 / FHWA (2011)

- Seismic environment (magnitude, frequency, duration) characterized by $\beta = S_1/PGA$
  - After correcting for local site conditions
- $\beta = 1.5$: Upper Bound, for large magnitude, west coast earthquakes
- $\beta = 0.5$: Lower Bound, for smaller magnitude east coast earthquakes
- $\beta = 1$: Intermediate value for intermediate events

NCHRP 12-70 / FHWA (2011)

Methodology

1. Adjust ground motions (PGA, S_1) for local site conditions
2. Adjust PGA for slope height, ground motion characteristics to get MHA
3. Adjust MHA based upon soil ductility to get k_s
4. Select FS based upon allowable displacement

NCHRP 12-70 / FHWA (2011)

1. Adjust ground motions for local site conditions, embankments
   - Method A: Site factors
   - Method B: Site specific hazard analysis that includes local site conditions
   - Method C: Reference site ground motions, site response analysis (e.g. SHAKE)

NCHRP 12-70 / FHWA (2011)

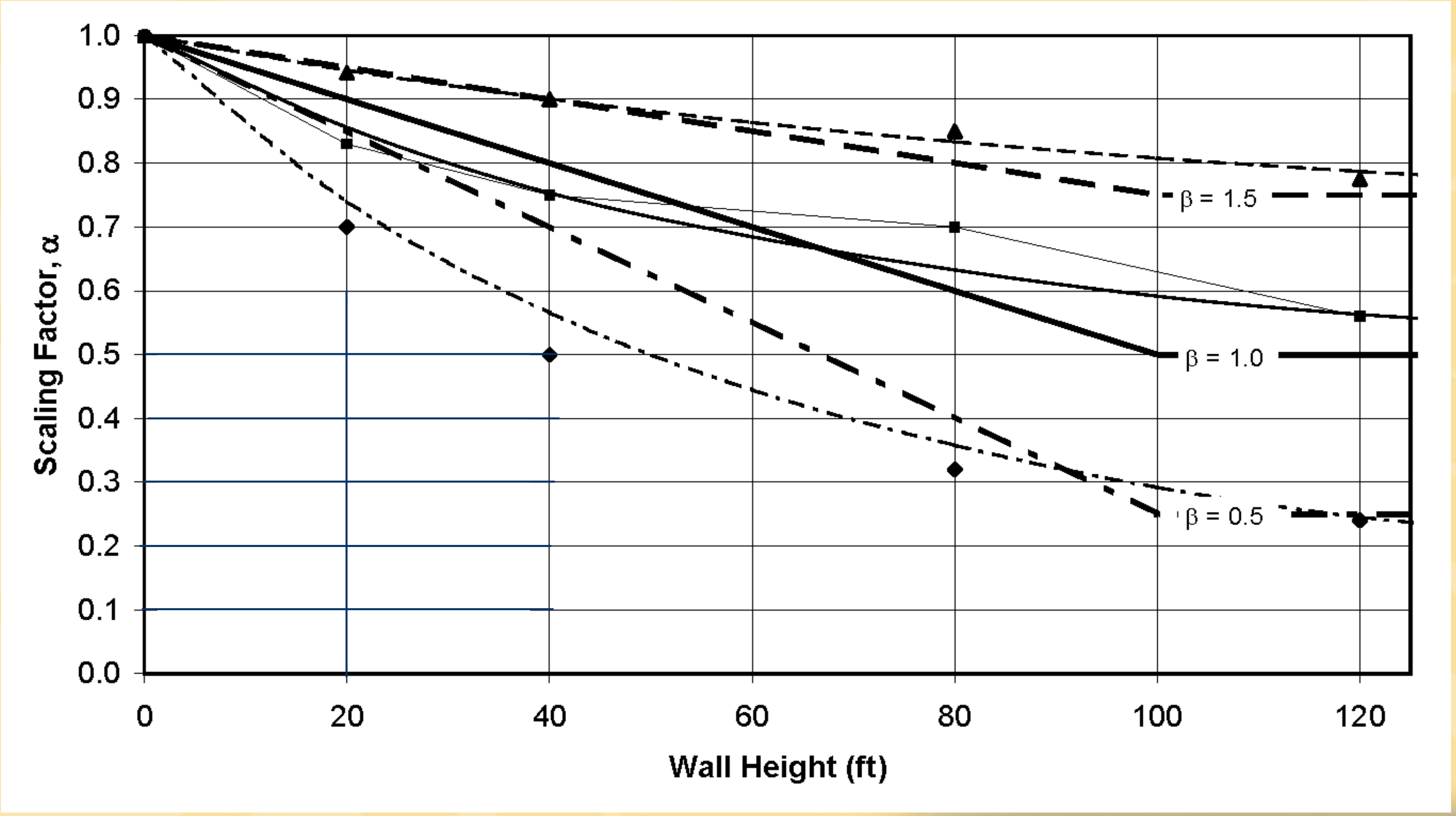
2. Adjust site corrected PGA for slope height, ground motion characteristics to get MHA

NCHRP / FHWA

MHA = α x PGA, $\alpha = 1 + 0.01H\,[0.5\beta - 1]$ (H ≤ 100 ft)

Notes: H is in feet

For rock sites (Site Class A and B), increase α by 20%



NCHRP 12-70 / FHWA (2011)

3. Adjust MHA for soil ductility to get k_s

$k_s = r \times (MHA/g) = r \times \alpha \times (PGA/g)$, where r = allowable displacement (ductility) factor

– r = 1 for brittle soil

– r = 0.5 for ductile soil

NCHRP 12-70 / FHWA (2011)

4. Establish FS based upon allowable displacement

For negligible displacements:

- If $r = 1$: $k_s = MHA/g = \alpha \times PGA/g$, $FS = 1.0$
- If $r = 0.5$: $k_s = 0.5 \times \alpha \times PGA/g$, $FS = 1.1$

For small displacements (5 cm max):

- $r = 0.5$: $k_s = 0.5 \times \alpha \times PGA/g$, $FS = 1.0$

NCHRP / FHWA SUMMARY

1. Find PGA and S_1 (include site effects)
2. Get $\beta = (S_1 \times F_V) / (PGA \times F_{PGA})$
3. Get $\alpha = 1 + 0.01\, H\, (0.5\beta - 1)$ or from chart
4. Find $k_s = r \times \alpha \times PGA$
   - For brittle system: r =1.0, $FS_{min} = 1.0$
   - For ductile system: r = 0.5
     - » $FS_{min} = 1.1$ for negligible displacement
     - » $FS_{min} = 1.0$ for small (≤ 5 cm) displacement,

Vancouver Example 1: Site Class C

1000 yr Ground Motions

Site Class C: (from NRC Canada website)

- PGA = 0.32; S_1 = 0.23; β = 0.23/0.32 = 0.72

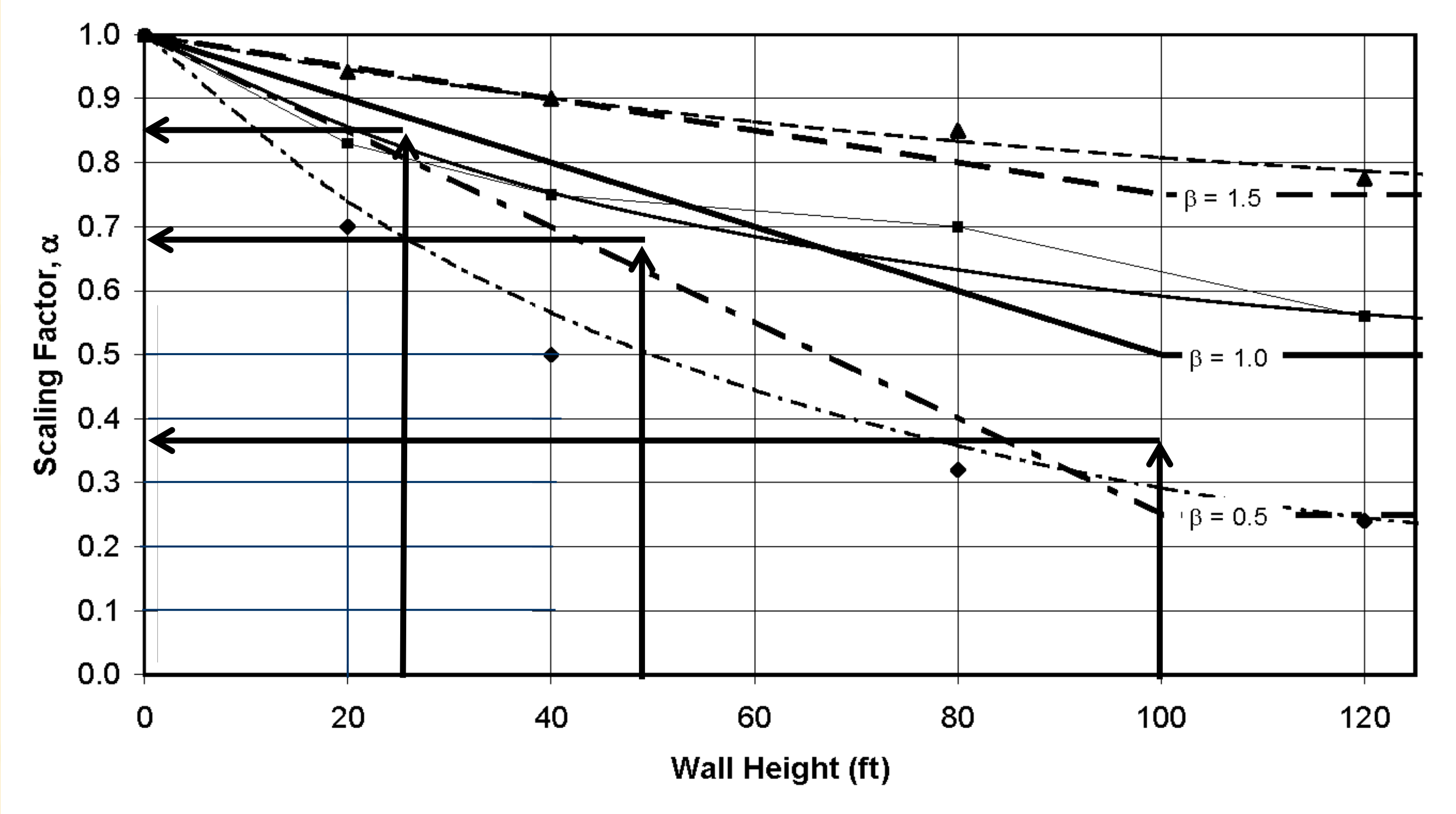
Find α:

- α = 0.85 @ H = 7.5 m, 0.69 @ H = 15 m, and 0.38 @ H = 30 m

Find k_s (for 5 cm displacement, i.e. r = 0.5):

- k_s = 0.14 for H = 7.5 m, 0.11 for H = 15 m, and 0.06 for H = 30 m

Vancouver Example 1



Vancouver Example 2: Site Class E

1000 yr Ground Motions

Site Class E: (from NRC Canada website)

- $(PGA)_C = 0.32$; $F_{PGA} = 1.06$; $(PGA)_E = 0.34$
- $(S_1)_C = 0.23$; $F_V = 1.97$; $(S_1)_E = 0.45$
- $\beta = 0.45/0.34 = 1.32$
- $\alpha = 0.92$ @ H = 7.5 m, 0.84 @ H = 15 m, and 0.68 @ H = 30 m

Find k_s (for 5 cm displacement, i.e. r = 0.5):

- $k_s = 0.16$ for H = 7.5 m, 0.14 for H = 15 m, and 0.12 for H = 30 m

Quebec Example 1: Site Class C

2500 yr Ground Motions

Site Class C: (from NRC website)

- PGA = 0.285; S_1 = 0.15; β = 0.15/0.28 = 0.54

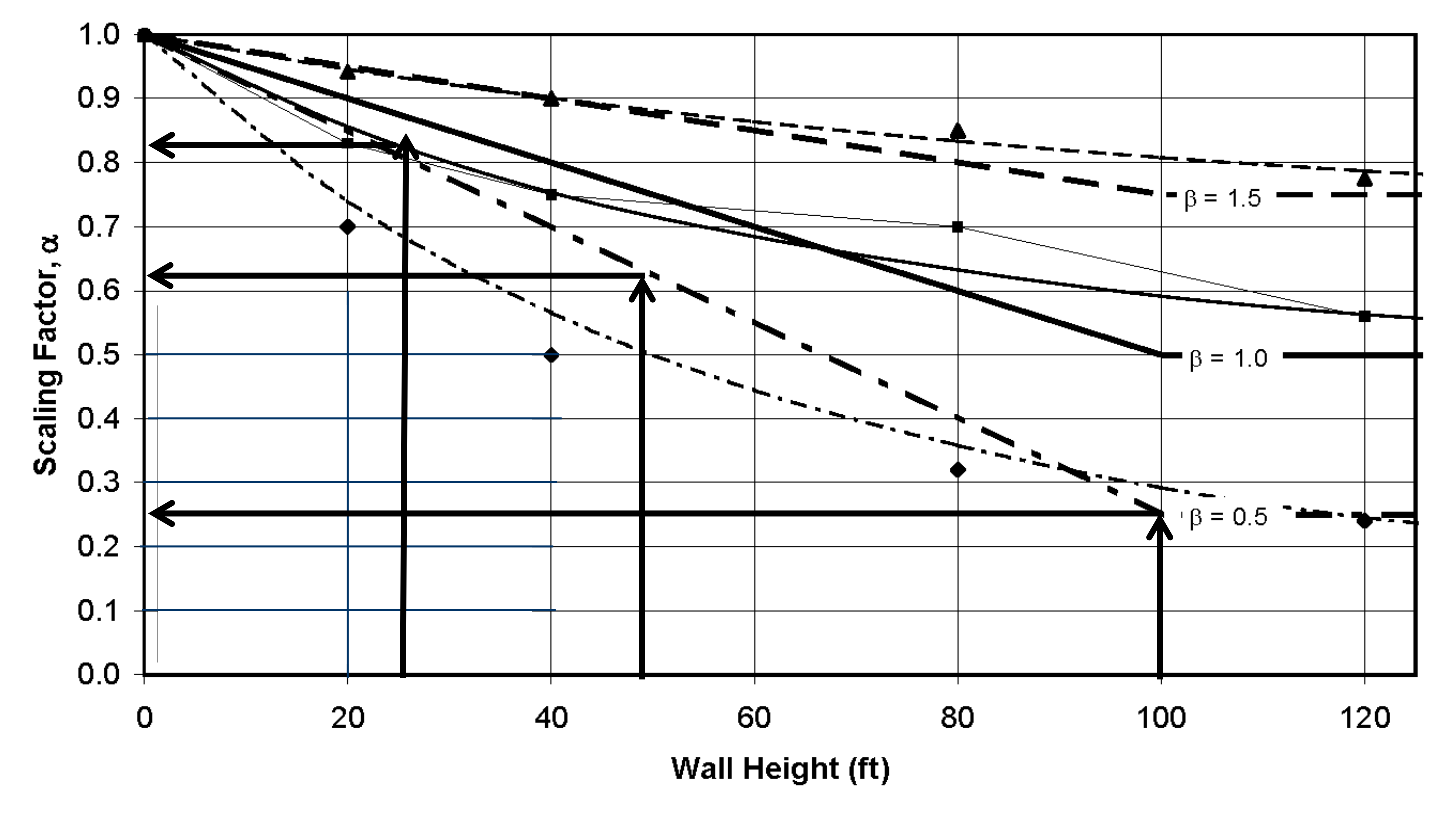
Find α:

- α = 0.82 @ 7.5 m, 0.63 @ 15 m, and 0.26 @ 30 m

Find k_s (for 5 cm displacement, i.e. r = 0.5):

- k_s = 0.12 for 7.5 m, 0.09 for 15 m, and 0.04 for 30 m

Quebec Example 1



Quebec Example 2: Site Class E

2500 yr Ground Motions

Site Class E: (from NRC website)

- $(PGA)_C = 0.285$; $F_{PGA} = 1.145$; $(PGA)_E = 0.33$
- $(S_1)_C = 0.15$; $F_V = 2.05$; $(S_1)_E = 0.31$
- $\beta = 0.31/0.33 = 0.\ 94$
- $\alpha = 0.87$ @ 7.5 m, 0.74 @ 15 m, and 0.49 @ 30 m

Find k_s (for 5 cm displacement, i.e. $r = 0.5$):

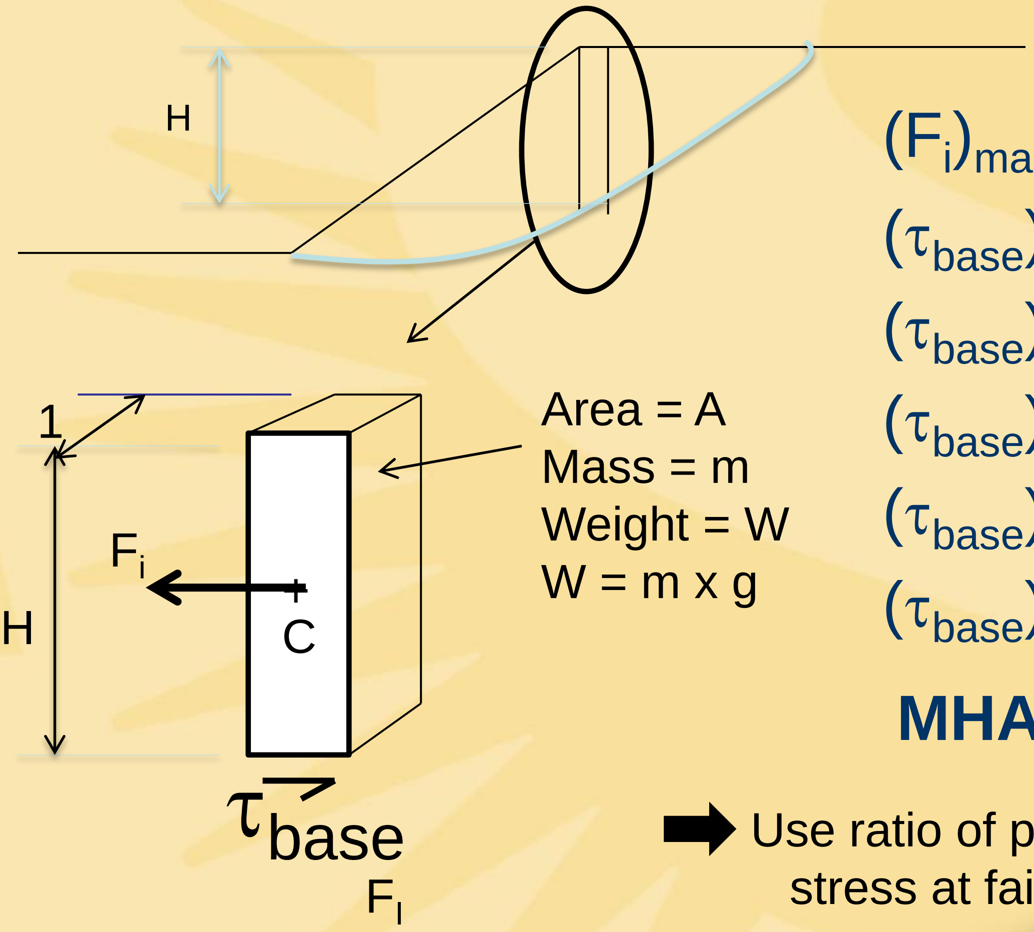
- $k_s = 0.14$ for 7.5 m, 0.12 for 15 m, and 0.08 for 30 m

Seismic Coefficient Examples Summary

Location	Site Class	Return Period (yrs)	PGA (g)	H = 7.5 m	H = 15 m	H = 30 m
Vancouver	C	1000	0.32	0.14	0.11	0.06
Vancouver	E	1000	0.34	0.16	0.14	0.12
Quebec	C	2500	0.29	0.12	0.09	0.04
Quebec	E	2500	0.33	0.14	0.12	0.08

MHA from Site Response Analysis

Method C: Site Response (e.g. SHAKE) Analysis



$(F_i)_{max}$ = m x MHA

$(\tau_{base})_{max} = F_{max}/A$

$(\tau_{base})_{max}$ = [m x MHA] / A

$(\tau_{base})_{max}$ = [(mxg)/A] x [MHA/g]

$(\tau_{base})_{max}$ = [W/A] x [MHA/g]

$(\tau_{base})_{max} = \sigma_v$ x [MHA/g]

$$\textbf{MHA} = [(\tau_{base})_{max}/\sigma_v]_{max} \times g$$

➡ Use ratio of peak shear stress to total vertical stress at failure plane elevation to get MHA

Bray and Travasarou (2009)

k_s based upon:

- Probabilistic equation for Newmark displacement (Bray and Travasarou 2007)
- Fundamental period of potential slide mass, T_s
  - $T_s = 4H/V_s$
- Spectral acceleration, S_a, at a spectral period = 1.5 x T_s (equal to degraded fundamental period)
- Allowable displacement

Bray and Travasarou (2009)

Ground motion characterization:

- Requires spectral acceleration at 1.5 x T_S
  - May need entire response spectrum
  - $T_S = 4H/V_S$
    - Only use $T_S = (2.6 \times H) / V_S$ for earth dams (triangular embankment)
- Also requires earthquake magnitude, M
  - Need to deaggregate hazard

Summary and Conclusions (6)

- NCHRP / FHWA well suited for relatively uniform profiles
  - No sharp impedance contrast in top 150 – 200 ft
- Formal response analysis (e.g. SHAKE) can be used with NCHRP method for all soil profiles
- Bray and Travasarou may be OK for layered profiles
  - Sharp impedance contrasts at base of slope

References

Bray, J. D., and Travasarou, T. (2007) "Simplified procedure for estimating earthquake-induced deviatoric slope displacements." J. Geotech. Geoenviron. Eng., ASCE, Vol. 133, No. 4, pp. 381–392.

Bray, J. D., and Travasarou, T. (2009) "Pseudostatic Coefficient for Use in Simplified Seismic Slope Stability Evaluation," J. Geotech. Geoenviron. Eng., ASCE, Vol. 135, No. 9, pp. 1336-1340

FHWA (1998) "Design Guidance: Geotechnical Earthquake Engineering for Highways," Geotechnical Engineering Circular No. 3, Federal Highway Administration, Vol. 1 and Vol. 2, Reports No. FHWA-SA-97-076 and FHWA-SA-97-077

FHWA (2011) "LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations," Geotechnical Engineering Circular No. 3, Report No. FHWA-NHI-11-032, 592 p"

References

Harder, L.F., Jr. (1991), "Performance of Earth Dams During the Loma Prieta Earthquake," Proc. Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, University of Missouri, Rolla, pp. 11-15.

Hynes, M.E. and Franklin, A.G. (1984). "Rationalizing the Seismic Coefficient Method," Miscellaneous Paper GL-84-13, U.S. Army Waterways Experiment Station, Vicksburg, MS, 21 pp.

Idriss, I.M. (1990), "Response of Soft Soil Sites During Earthquakes," Proc. Memorial Symposium to Honor Professor H.B. Seed, Berkeley, California, Bi-Tech Publishers.

Kavazanjian, E., Jr. (1998), "Current Issues in Seismic Design of Geosynthetic Cover Systems," Proc. *Sixth International Conference on Geosynthetics,* Atlanta, Georgia, Vol. I, pp. 219-226

Makdisi, F.I. and Seed, H.B. (1978), "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations," J. Geotech. Eng. Division, ASCE, Vol. 104, No. GT7, pp. 849-867.

References

NCHRP 12-70 (2008) “Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes and Embankments,” NCHRP Report 611, Transportation Research Board, Washington, DC (2 volumes)

NEHRP (1997) “Recommended Provisions for Seismic Regulations for New Buildings and Other Structures: Part 1, Provisions (FEMA 302),“ prepared for the Federal Emergency Management Agency by the Building Seismic Safety Council, Washington, DC

Seed, H. B. 1979. “Considerations in the Earthquake-Resistant Design of Earth and Rockfill Dams.” Geotechnique, 293, 215–263.

Seed, H.B. and Idriss, I.M. (1982), "Ground Motions and Soil Liquefaction During Earthquakes," Monograph No. 5, Earthquake Engineering Research Institute, Berkeley, California, 134 p.

Questions?

Effect of Vertical Acceleration