GEOTECHNICAL EARTHQUAKE ENGINEERING

Typically concerned with:

- *Determining ground motions* – especially as to effects of local site conditions
- *Liquefaction and liquefaction-related evaluations* – (settlements, lateral spreading movements, etc.)
- Slope/landslide evaluation
- Dams/embankments
- Design of retaining structures
- Deep and shallow foundation analysis
- Underground structures (tunnels, etc.)

Key Reference

Kramer, Steven L. 1996. *Geotechnical Earthquake Engineering.* Prentice Hall, 653 pp.

Historical Perspective

"While many cases of soil effects had been observed and reported for many years, it was not until a series of catastrophic failures, involving landslides at Anchorage, Valdez and Seward in the 1964 Alaska earthquake, and extensive liquefaction in Niigata, Japan, during the earthquake in 1964, caused geotechnical engineers to become far more aware of, and eventually engaged in understanding, these phenomena."

(I. M. Idriss, 2002)

Important Learning Opportunities

- 1964 Niigata and 1964 Alaska
- 1967 Caracas
- 1971 San Fernando
- 1979 Imperial valley
- 1985 Mexico City
- 1989 Loma Prieta
- 1995 Kobe (Japan)
- 1999 Kocaeli (Turkey)
- 1999 Chi Chi (Taiwan)

Site Effects – Some History

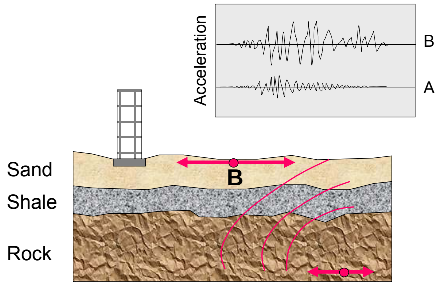
"… a movement … must be modified while passing through media of different constitutions. Therefore, the earthquake effects will arrive to the surface with higher or lesser violence according to the state of aggregation of the terrain which conducted the movement. This seems to be, in fact, what we have observed in the Colchagua Province (of Chile) as well as in many other cases."

- from Del Barrio (1855) in Toro and Silva (2001)

Site Effects on Ground Motions

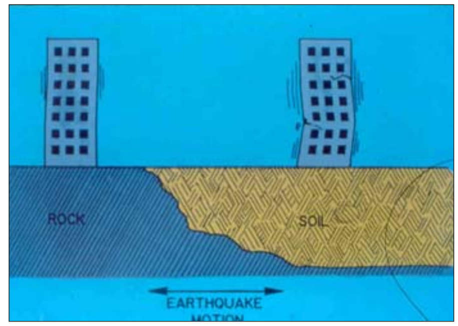
- Soil profile acts as filter
- Change in frequency content of motion
- Layering complicates the issue
- Amplification or de-amplification of ground motions can occur
- Duration of motion is increased

Site Amplification Is Common

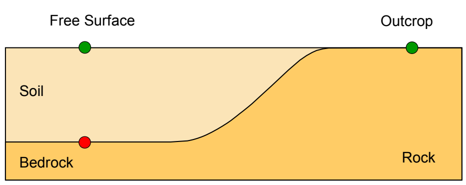


Site Effects on Ground Motions

Conservation of energy drives amplification



Amplification Definitions



Amplification = $\frac{\text{Free Surface}}{\text{Bedrock}}$ Amplification = $\frac{\text{Free Surface}}{\text{Outcrop}}$

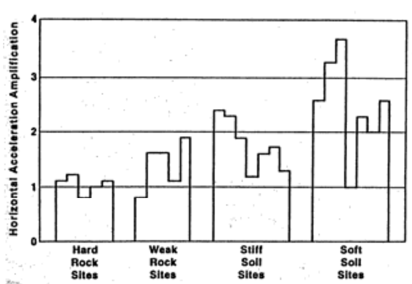
Figure adapted from Rix, G. J., (2001)

Amplification Definitions

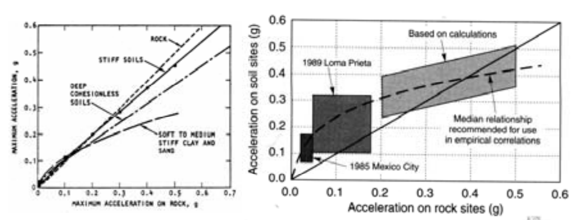
- Fourier amplification spectra
- Spectral amplification

$$\left|\frac{a_{\text{free surface}}(f)}{a_{\text{outcrop}}(f)}\right| \qquad \left|\frac{S_{a,\text{ free surface}}(T)}{S_{a,\text{ outcrop}}(T)}\right|$$

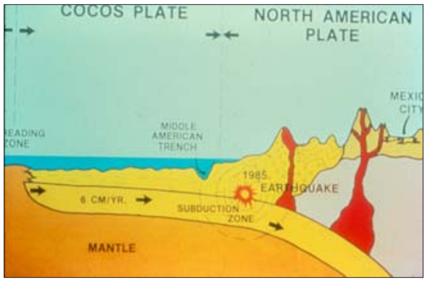
Soft Soils Commonly Amplify Motions Relative To Bedrock



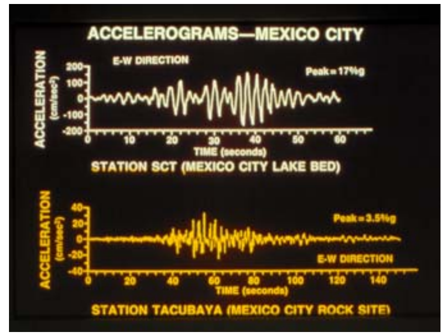
Effects of Local Soil Conditions



1985 Mexico City Earthquake

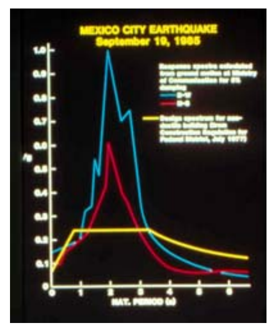


1985 Mexico City Accelerograms



1985 Mexico City – Juarez Hospital

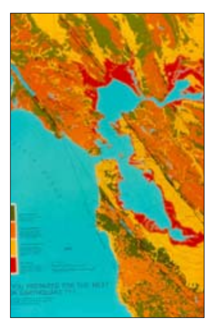
1985 Mexico City – Response Spectra



1989 Loma Prieta Earthquake

San Francisco Bay Geological Map

- Soft deposits in red (Bay mud)



San Francisco Marina District

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Damage in Marina District

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Cypress Structure Collapse

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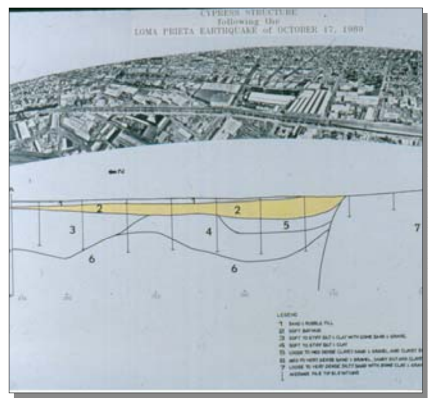

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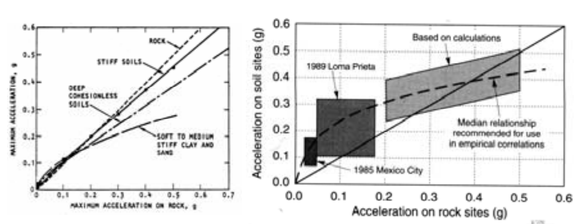
Cypress Structure Collapse

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Geotechnical 15-4 - 23

Effects of Local Soil Conditions



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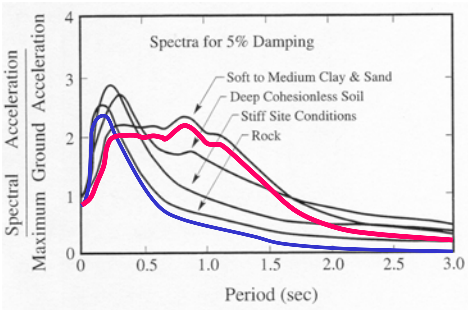
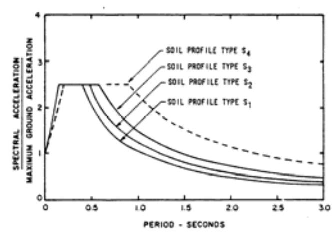
Effects of Local Soil Conditions



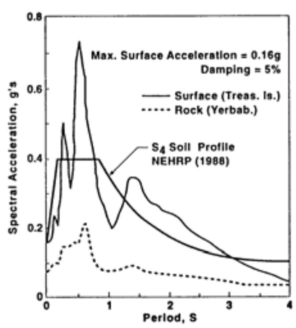
Pre-Loma Prieta Design Spectra



Soil Factor, S (NEHRP, 1988)

Type	Description	NEHRP
S_1	A soil profile with either: (a) A rock-like material characterized by a shear-wave velocity greater than 2,500 feet per second or by other suitable means of classification, or (b) stiff or dense soil condition where the soil depth is less than 200 feet.	1.0
S_2	A soil profile with dense or stiff soil conditions, where the soil depth exceeds 200 feet or more.	1.2
S_3	A soil profile 70 feet or more in depth and containing more than 20 feet of soft to medium stiff clay but not more than 40 feet of soft clay.	1.5
S_4	A soil profile, characterized by a shear wave velocity less than 500 feet per second, containing more than 40 feet of soft clay.	2.0

Spectrum from 1989 Loma Prieta at Deep Soft Soil Site



Reason for F Category in IBC 2003

IBC2003 – "F" Requires Site-specific Analysis

TABLE 1615.1.2(1)
VALUES OF SITE COEFFICIENT F_a AS A FUNCTION OF SITE CLASS AND MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS (S_S)[a]

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	Note b
F	Note b	Note b	Note b	Note b	Note b

a. Use straight line interpolation for intermediate values of mapped spectral acceleration at short period, S_S.
b. Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values.

TABLE 1615.1.2(2)
VALUES OF SITE COEFFICIENT F_V AS A FUNCTION OF SITE CLASS AND MAPPED SPECTRAL RESPONSE ACCELERATION AT 1 SECOND PERIOD (S_1)[a]

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT 1 SECOND PERIOD				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	Note b
F	Note b	Note b	Note b	Note b	Note b

a. Use straight line interpolation for intermediate values of mapped spectral acceleration at 1-second period, S_1.
b. Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values.

IBC2003 – "F" Requires Site-specific Analysis

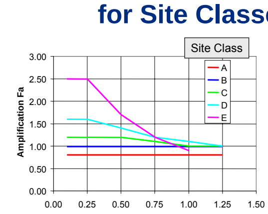
- Determine site class based on top 30 m:

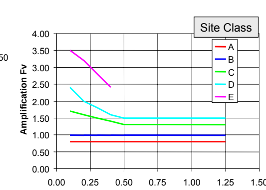
TABLE 1615.1.1
SITE CLASS DEFINITIONS

SITE CLASS	SOIL PROFILE NAME	AVERAGE PROPERTIES IN TOP 100 feet, AS PER SECTION 1615.1.5		
		Soil shear wave velocity, $\bar{v}_s$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{s}_u$, (psf)
A	Hard rock	$\bar{v}_s > 5{,}000$	Not applicable	Not applicable
B	Rock	$2{,}500 < \bar{v}_s \le 5{,}000$	Not applicable	Not applicable
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \le 2{,}500$	$\bar{N} > 50$	$\bar{s}_u \ge 2{,}000$
D	Stiff soil profile	$600 \le \bar{v}_s \le 1{,}200$	$15 \le \bar{N} \le 50$	$1{,}000 \le \bar{s}_u \le 2{,}000$
E	Soft soil profile	$\bar{v}_s < 600$	$\bar{N} < 15$	$\bar{s}_u < 1{,}000$
E	—	Any profile with more than 10 feet of soil having the following characteristics: 1. Plasticity index $PI > 20$; 2. Moisture content $w \ge 40\%$, and 3. Undrained shear strength $\bar{s}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ ft)		

For SI: 1 foot = 304.8 mm, 1 square foot = 0.0929 m², 1 pound per square foot = 0.0479 kPa.

NEHRP Provisions Site Amplification for Site Classes A through E

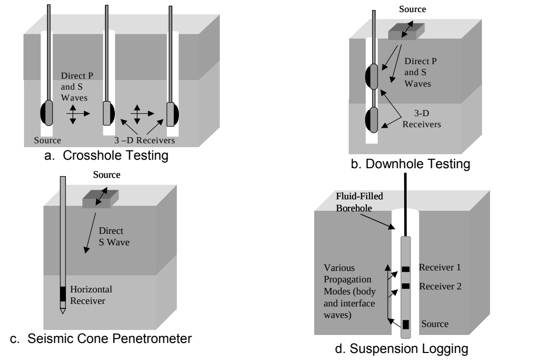




Site Classification from?

- *NEHRP Provisions* allow site classification to determined from various geotechnical data, such as SPT blowcounts, undrained shear strength, and shear wave velocity measurements (V_s)
- Best approach $\Rightarrow$ in situ V_s measurement

Field Tests To Measure Seismic Wave Velocities

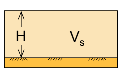


Courtesy of K. H. Stokoe II

Site Response Mechanisms

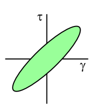
- Constant flux rate – impedance

  $\rho V_s \dot{u}^2$ = constant
- Resonances within the soil column



$$f_n = \frac{V_s}{4H}$$

- Low-strain damping and apparent attenuation in soil
- Nonlinear soil behavior



Amplification

Deamplification

Figure adapted from Rix, G. J., (2001)

Site Response Analysis - Two Steps

(1) Modeling the soil profile

(2) Calculating the site-modified time histories or other motions at various level within the profile, typically, at the ground surface

(1) Modeling the Soil Profile

- The stratigraphy and dynamic properties (dynamic moduli and damping characteristics) of the soil profile are modeled.
- If soil depth is reasonably constant beneath the structure and the soil layers and ground surface reasonably flat, then a one-dimensional analysis can be used.
- Two- or three-dimensional models of the site can be used where above conditions are not met.
- Unless soil properties are well constrained a range of properties should be defined for the soil layers to account for uncertainties.

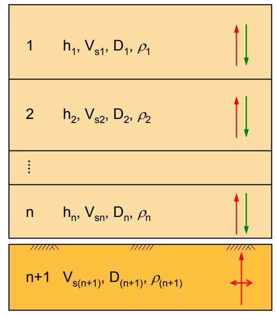
(2) Calculating top-of-profile motions:

- Typically the design bedrock time-histories are input to the soil model and the corresponding top-of-soil time-histories are obtained.
- Analysis should incorporate nonlinear soil behavior either through the equivalent linear method or true nonlinear analysis methods.
- Ensure program properly accounts for motion recorded on outcrop being input at base, etc.
- Issue: where to assume base or halfspace? (V_s = 2000 fps is often assumed but not always OK)

Site Response Analysis Techniques

- Linear analyses
- Quarter-wavelength approximation
- Equivalent linear analyses
- Nonlinear analyses

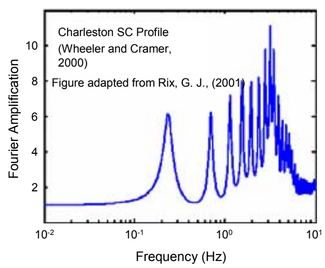
Site Response Calculations



- Layered profile
- Vertically propagating, horizontally polarized shear waves
- Calculate the amplitude of up-going and down-going waves in each layer by enforcing the compatibility of displacements and stresses at layer interface

Figure adapted from Rix, G. J., (2001)

Linear Analysis



- Constant V_s (i.e., G) and D (i.e., Q)
- Amplification from Pre-Cretaceous outcrop (hard rock) to ground surface. Soil profile is ~1 km thick.

Equivalent-Linear Analysis (i.e., SHAKE)

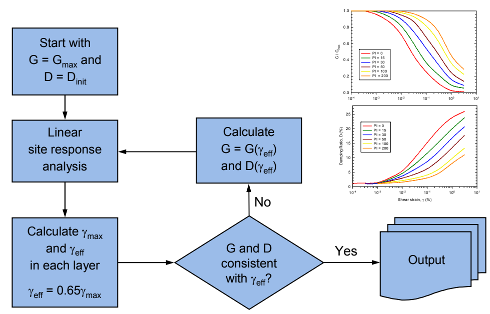
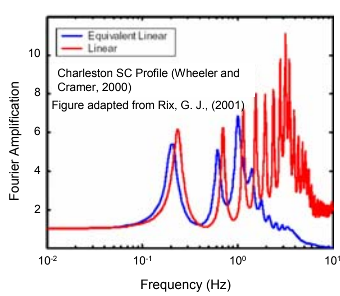
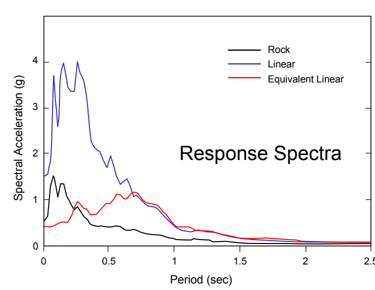


Figure adapted from Rix, G. J., (2001)

Equivalent Linear Analysis

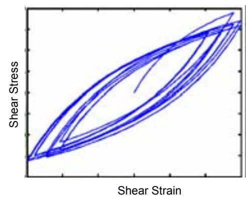


Equivalent Linear Analysis



Nonlinear Analysis

- Choose a constitutive model representing nonlinear cyclic soil behavior (nonlinear inelastic, cyclic plasticity, pore pressure generation)
- Integrate the equation of motion for vertically propagating shear waves in time domain
- Programs available are DESRA, FLAC, DYNAFLOW, SUMDES, etc.



Equivalent Linear vs. Nonlinear

- The inherent linearity of equivalent linear analyses can lead to "spurious" resonances.
- The use of effective shear strain can lead to an over-softened and over-damped system when the peak shear strain is not representative of the remainder of the shear-strain time history and vice versa.
- Nonlinear methods can be formulated in terms of effective stress to model generation of excess pore pressures.
- Nonlinear methods require a robust constitutive model that may require extensive field and lab testing to determine the model parameters.
- Difference between equivalent linear and nonlinear analyses depend on the degree of nonlinearity in the soil response. For low to moderate strain levels (i.e. weak input motions and/or stiff soils), equivalent linear methods provide satisfactory results.

-- from Kramer (1996)

Site Response Analysis Codes

A. One-dimensional equivalent-linear codes:

- SHAKE (Schnabel, Seed, and Lysmer 1972; Idriss and Sun 1992)
- WESHAKE (Sykora, Wahl, and Wallace 1992);

Site Response Analysis Codes

B. One-dimensional nonlinear codes:

- DESRA-2 (Lee and Finn 1978), DESRA-MUSC (Qiu 1998)
- SUMDES (Li, Wang, and Shen 1992)
- MARDES (Chang et al. 1990)
- D-MOD (Matasovic 1993)
- TESS (Pyke 1992)

Site Response Analysis Codes

C. 2-D and 3-D equivalent linear codes:

- FLUSH (2-D) (Lysmer et al. 1975)
- QUAD4M (Hudson, Idriss, and Beikae 1994)
- SASSI (2-D or 3-D) (Lysmer et al. 1991)

Dynamic Soil Properties

- Shear wave velocity profile

$$G_{max} = \rho \cdot V_s^2$$

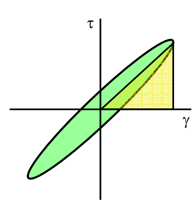
- Nonlinear soil behavior

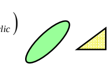
Modulus reduction curve

$$G_{sec} / G_{max} = f(\gamma_{cyclic})$$

Material damping ratio curve

$$D = \frac{1}{4\pi}\frac{\Delta W}{W} = f(\gamma_{cyclic})$$





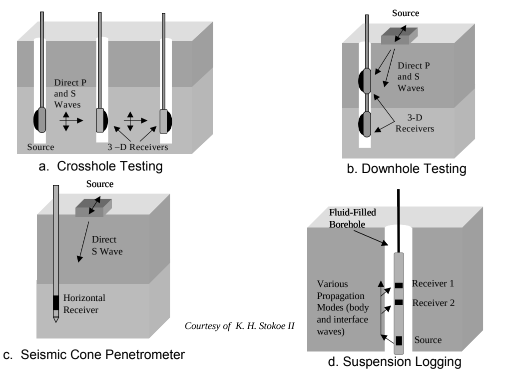
Laboratory Methods

- Resonant column
- Torsional shear
- Cyclic simple shear
- Cyclic triaxial
- Bender elements

In Situ Methods

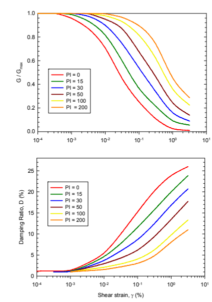
- Invasive methods
  - Crosshole
  - Downhole/SCPT
  - P-S suspension logger
- Invasive methods for nonlinear soil properties
- Vertical arrays
- Noninvasive methods
  - Refraction
  - High-resolution seismic reflection
  - Surface wave methods
- Empirical correlations with SPT and CPT

In Situ Tests to Measure Seismic Wave Velocities



Courtesy of K. H. Stokoe II

Modulus Reduction and Damping



Vucetic and Dobry (1991)

- Seed et al. (1986)
- Sun et al. (1988)
- Ishibashi and Zhang (1993)
- EPRI (1993)
- Hwang (1997)
- Assimaki et al. (2000)
- Toro and Silva (2001)

Liquefaction

"If a saturated sand is subjected to ground vibrations, it tends to compact and decrease in volume.

If drainage is unable to occur, the tendency to decrease in volume results in an increase in pore pressure.

If the pore water pressure builds up to the point at which it is equal to the overburden pressure, the effective stress becomes zero, the sand loses its strength completely, and liquefaction occurs."

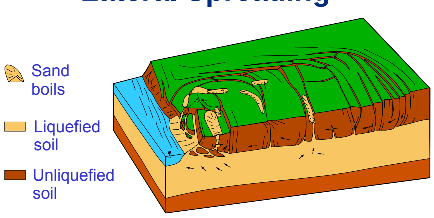
Seed and Idriss

Liquefaction - Field of Sand Boils

Liquefaction Damage, Niigata, Japan, 1964

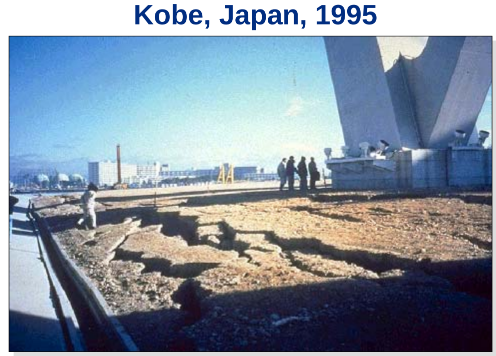
Liquefaction Damage, Adapazari, Turkey, 1999

Lateral Spreading

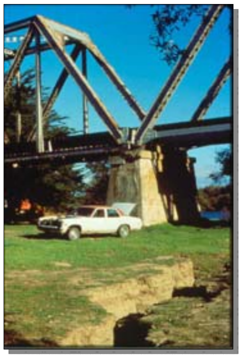


- Mostly horizontal deformation of gently-sloping ground (< 5%) resulting from soil liquefaction
- One of most pervasive forms of ground damage; especially troublesome for lifelines

Liquefaction and Lateral Spreading, Kobe, Japan, 1995



Lateral Spreading, Loma Prieta, 1989



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Pile Damage Beneath Building by Lateral Spread 1964, Niigata, Japan

Photo courtesy of Professor T. L. Youd from Elgamal (2002)

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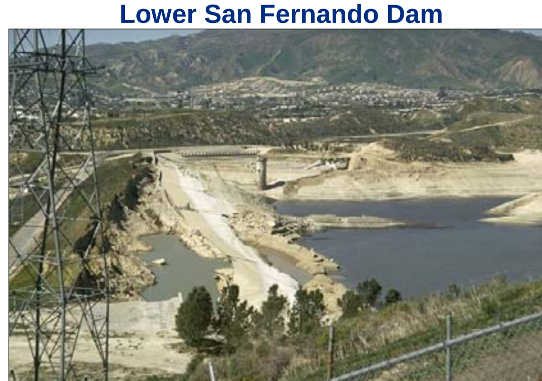
Lower San Fernando Dam

Instructional Material Complementing FEMA 451, *Design Examples* Geotechnical 15-4 - 63

Lower San Fernando Dam



Instructional Material Complementing FEMA 451, *Design Examples* Geotechnical 15-4 - 64

Liquefaction Damage

- In the 1994 Northridge earthquake, homes damaged by liquefaction or ground failure were 30 times more likely to require demolition than those homes only damaged by ground shaking (ABAG)
- In the 1995 Kobe Japan Earthquake, significant damages occurred to port facilities due to liquefaction; after almost 10 years post trade still 10-15% off

Instructional Material Complementing FEMA 451, *Design Examples* Geotechnical 15-4 - 65

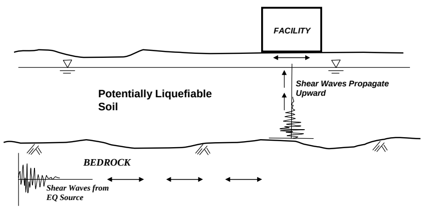
Key Reference

Youd et al. 2001. "Liquefaction Resistance Of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," *Journal of Geotechnical and Geoenvironmental Engineering,* October, pp. 817-833.

Instructional Material Complementing FEMA 451, *Design Examples* Geotechnical 15-4 - 66

Liquefaction Analysis

Saturated loose sands, silty sands, sandy silts, nonplastic silts, and some gravels are susceptible to liquefaction in an earthquake.



Liquefaction Analysis

- A quantified measure of seismically induced shaking within a soil profile is termed the earthquake demand. The most commonly used measure of demand in current practice is the c*yclic stress ratio* (**CSR**).
- The soil's ability to resist this shaking without liquefaction is determined by one or more methods, and is indicated by its *cyclic resistance ratio* (**CRR**).

Liquefaction Analysis Steps

Step 1 -- Estimate the maximum acceleration at the ground surface, a_{max}:

This can be obtained from: (a) an actual acceleration record from nearby; (b) from "attenuation" relationships that relate a_{max} to the earthquake magnitude and include the effects of soil directly; (c) from a site response analysis using a series of time histories (if this is done, CSR can be determined directly from the output); (d) soft soil amplification factors such as Idriss (1990); and (e) national seismic hazard maps.

Liquefaction Analysis

Step 2 -- Determine the cyclic shear stress ratio, CSR, according to:

$$CSR = \frac{\tau_{ave}}{\sigma_{vo}'} = 0.65 \frac{a_{max}}{g} \frac{\sigma_{vo}}{\sigma_{vo}'} r_d$$

in which

τ_{ave} = average cyclic shear stress

σ'_{vo} = vertical effective stress (total vertical stress minus the pore water pressure) at the depth of interest

σ_{vo} = total vertical stress at the depth of interest

g = acceleration due to gravity

r_d = depth reduction factor (see Figure 1)

Figure 1 – R_d vs. Depth

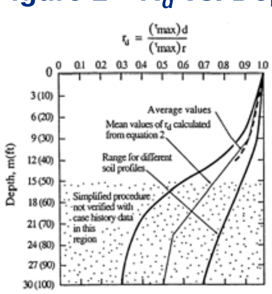


Fig. 1. r_d Versus Depth Curves Developed by Seed and Idriss (1971) with Added Mean Value Lines Plotted from Eq. 2

Liquefaction Analysis

Step 3 -- Determine the soil resistance to liquefaction, CRR.

CRR can be determined from the results of Standard Penetration Tests (SPT) – see Figure 2, Cone Penetration Tests (CPT) – see Figure 3, or Shear Wave Velocity Measurements (V_s) - see Figure 4, may be used. Characteristics and comparisons of these test methods are given in Table 1.

⇒ The SPT N-value method is described here for level ground.

Figure 2- $N_{1,60}$ vs. CSR/CRR

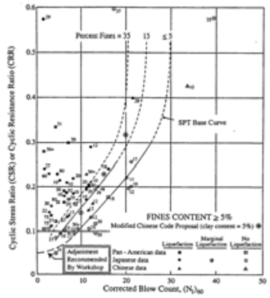


Fig. 2. Base Curve for Magnitude 7.5 Earthquakes Recommended for Calculation of CRR from SPT Data with Data From Compiled Liquefaction Case Histories (modified From Seed et al., 1985)

Liquefaction Analysis

Step 4 -- Determine SPT N-values at several depths over the range of interest. These values must be corrected to account for depth (overburden pressure) and several other factors as listed in Table 2 to give the normalized penetration resistance $(N_1)_{60}$ which corresponds to a hammer efficiency of 60%.

$$(N_1)_{60} = N \cdot C_N \cdot C_E \cdot C_B \cdot C_R \cdot C_S$$

where:

- N = measured penetration resistance, blows per foot
- C_N = correction for overburden pressure = $(P_a/\sigma'_{vo})^{0.5}$
- P_a = atmospheric pressure in same units as σ'_{vo} = 1 tsf, 100 kPa, 1 kg/cm^2
- C_E = energy correction (see Table 2)
- C_B = borehole diameter correction (see Table 2)
- C_R = correction for rod length (see Table 2)
- C_S = correction for sampling method (see Table 2)

Table 2. SPT Correction Factors

Factor	Test Variable	Term	Correction
Overburden Pressure[1]		C_N	$(P_a/\sigma_{vo}')^{0.5}$ $C_N \le 1.7$
Energy Ratio	Donut Hammer Safety Hammer Automatic-Trip DonutType Hammer	C_E	0.5 to 1.0 0.7 to 1.2 0.8 to 1.3
Borehole Diameter	65 mm to 115 mm 150 mm 200 mm	C_B	1.0 1.05 1.15
Rod Length[2]	< 3 m 3 m to 4 m 4 m to 6 m 6 m to 10 m 10 m to 30 m > 30 m	C_R	0.75 0.8 0.85 0.95 1.0 >1.0
Sampling Method	Standard Sampler Sampler without Liners	C_S	1.0 1.1 to 1.3

Liquefaction Analysis

Step 5 -- Locate $(N_1)_{60}$ on Figure 2. If the earthquake magnitude is 7.5 and the depth of the point being evaluate corresponds to an effective overburden pressure of 1 tsf, 100 kPa, or 1 kg/cm^2, then the cyclic resistance ratio (CRR) is given by the corresponding value from the curve that separates the zones of liquefaction and no liquefaction (note that the appropriate curve to use depends on the fines content of the soil).

Liquefaction Analysis

Step 6 -- If the effective overburden pressure (σ'_{vo}) is greater than 1 tsf, 100 kPa or 1 kg/sq. cm, then the CRR should be reduced according to Figure 5 by:

$$(CRR)_{(\sigma'vo)} = (CRR)_{(\sigma'vo)=1} \times K_\sigma$$

If the earthquake magnitude is less than 7.5, then the CRR should be increased according to:

$$(CRR)_{M<7.5} = (CRR)_{M=7.5} \times MSF$$

The Magnitude Scaling Factor (MSF) is given by the shaded zone in Figure 6. Similarly, if the magnitude is greater than 7.5, then the CRR should be reduced according to the relationship in Figure 4.

Liquefaction Analysis

Step 7 --If the soil contains more than 5% fines, Fines content (FC corrections for soils with >5% fines may be made using (with engineering judgment and caution) the following relationships. $(N_1)_{60cs}$ is the clean sand value for use with base curve in Fig. 2.

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60}$$

$\alpha = 0$	for FC ≤ 5%
$\alpha = \exp[1.76 - (190/FC^2)]$	for 5% ≤ FC ≤ 35%
$\alpha = 5.0$	for FC ≥ 35%
$\beta = 1.0$	for FC ≤ 5%
$\beta = [0.99 + (FC^{1.5}/1000)]$	for 5% ≤ FC ≤ 35%
$\beta = 1.2$	for FC ≥ 35%

Liquefaction Analysis

Step 8 -- The factor of safety against liquefaction is defined by:

$$\boxed{FS_{LIQ'N} = CRR/CSR}$$

Typically want FS >1.35 or so.

Table 1- Comparison of In Situ Tests

Feature	Test Type			
	SPT	CPT	V_s	BPT
Data base from past EQ's	Abundant	Abundant	Limited	Sparse
Type of stress-deformation in test	Partly drained, large strain	Drained, large strain	Small strain	Partly drained, large strain
Quality control, repeatability	Poor to good	Very good	Good	Poor
Detection of heterogeneity	Good if tests closely spaced	Very good	Fair	Fair
Most suitable soil types	Gravel free	Gravel free	All	Gravelly soil
Soil sample obtained	Yes	No	No	Possibly
Index value or property measured directly	Index	Index	Property	Index
Data suitable for theoretical interpretation/analysis	No	Yes	Yes	No

Figure 3 - CPT vs. CSR/CRR

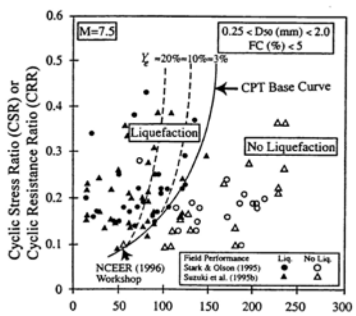


Figure 4 - Shear Wave Velocity

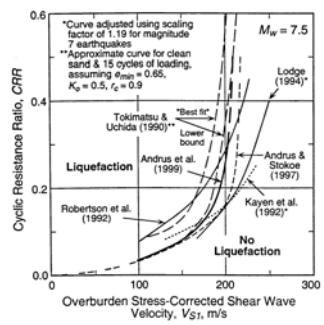


Figure 5 - Recommended Factors for Kσ

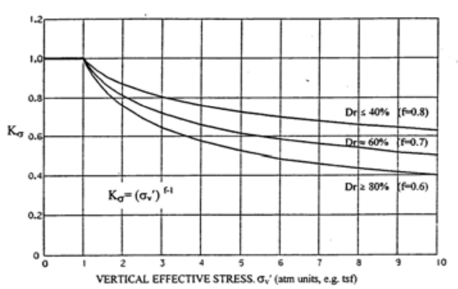
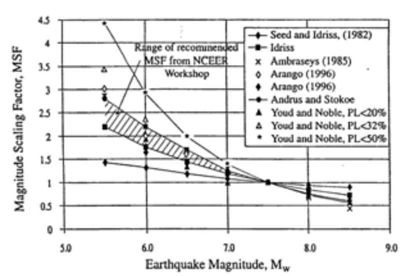


Figure 6 - Magnitude Scaling Factors



Soils With Plastic Fines: Chinese Criteria

Clayey Sands

Potentially liquefiable clayey soils need to meet all of the following characteristics (Seed et al., 1983):

•Percent finer than 0.005 mm < 15
•Liquid Limit (LL) < 35
•Water content > 0.9 x LL

If soil has these characteristics (and plot above the A-Line for the fines fraction to be classified as clayey), cyclic laboratory tests may be required to evaluate liquefaction potential. Recent work suggests latter two criteria work well to distinguish liquefiable soil, but the criterion of "percent finer than 0.005" does not match recent field experience (Martin et al., 2004).

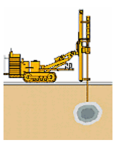
Liquefaction Remediation

- Basic approach is to either increase capacity (i.e., increase density, bind particles together), or decrease demand (i.e., soil reinforcement)
- Recent studies indicate cost/benefit ratio of liquefaction and site remediation is generally > 1.0
- Excellent summary of performance and techniques available from:

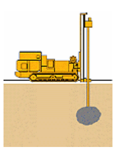
http://www.ce.berkeley.edu/~hausler/home.html

Liquefaction Remediation – Brief Summary

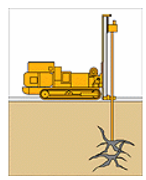
Source of following slides: http://www.haywardbaker.com/



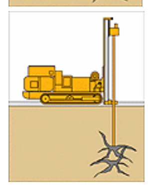
Compaction Grouting
When low-slump compaction grout is injected into granular soils, grout bulbs are formed that displace and densify the Surrounding loose soils. The technique is ideal for remediating or preventing structural settlements, and for site improvement of loose soil strata.



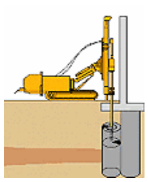
Chemical Grouting
The permeation of very low-viscosity chemical grout into granular soil improves the strength and rigidity of the soil to limit ground movement during construction. Chemical grouting is used extensively to aid soft ground tunneling and to control groundwater intrusion. As a remedial tool, chemical grouting is effective in waterproofing leaking subterranean structures.



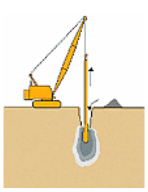
Cement Grouting Primarily used for water control in fissured rock, Portland and microfine cement grouts play an important role in dam rehabilitation, not only sealing water passages but also strengthening the rock mass. Fast-set additives allow cement grouting in moving water and other hard-to-control conditions.



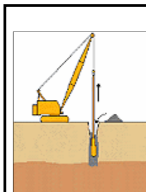
Soilfrac Grouting Soilfrac^{sm} grouting is used where a precise degree of settlement control is required in conjunction with soft soil stabilization. Cementitious or chemical grouts are injected in a strictly controlled and monitored sequence to fracture the soil matrix and form a supporting web beneath at-risk structures.



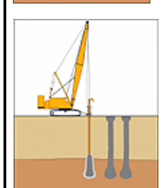
Jet Grouting Jet grouting is an erosion/replacement system that creates an engineered, in situ soil/cement product known as Soilcrete^{sm}. Effective across the widest range of soil types, and capable of being performed around subsurface obstructions and in confined spaces, jet grouting is a versatile and valuable tool for soft soil stabilization, underpinning, excavation support and groundwater control.



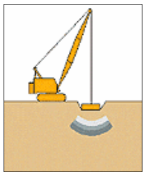
Vibro-Compaction A site improvement technique for granular material, Vibro-Compaction uses company-designed probe-type vibrators to densify soils to depths of up to 120 feet. Vibro-Compaction increases bearing capacity for shallow-footing construction, reduces settlements and also mitigates liquefaction potential in seismic areas.



Vibro-Replacement Related to Vibro-Compaction, Vibro-Replacement is used in clays, silts, and mixed or stratified soils. Stone backfill is compacted in lifts to construct columns that improve and reinforce the soil strata and aid in the dissipation of excess pore water pressures. Vibro-Replacement is well suited for stabilization of bridge approach soils, for shallow footing construction, and for liquefaction mitigation.

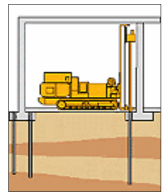


Vibro Concrete Columns Very weak, cohesive and organic soils that are not suitable for standard Vibro techniques can be improved by the installation of Vibro Concrete Columns. Beneath large area loads, Vibro Concrete Columns reduce settlement, increase bearing capacity, and increase slope stability.



Dynamic Deep Compaction Dynamic Deep Compaction[tm] is an economic site improvement technique used to treat a range of porous soil types and permit shallow, spread footing construction. Soils are densified at depth by the controlled impact of a crane-hoisted, heavy weight (15-35 tons) on the ground surface in a pre-determined grid pattern. Dynamic Deep Compaction is also successful in densifying landfill material for highway construction or recreational landscaping.

Soil Mixing Typically used in soft soils, the soil mixing technique relies on the introduction of an engineered grout material to either create a soil-cement matrix for soil stabilization, or to form subsurface structural elements to support earth or building loads. Soil mixing can be accomplished by many methods, with a wide range of mixing tools and tool configurations available.



Minipiles Underpinning of settling or deteriorating foundations, and support of footings for increased capacity are prime candidates for minipile installation, particularly where headroom is limited or access restricted. These small diameter, friction and/or end bearing elements can transfer ultimate loads of up to 350 tons to a competent stratum.

Extensive literature is available at the Hayward Baker Web-site: http://www.haywardbaker.com/

Vibrocompaction/Vibroreplacement

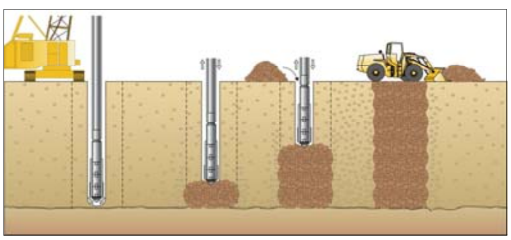
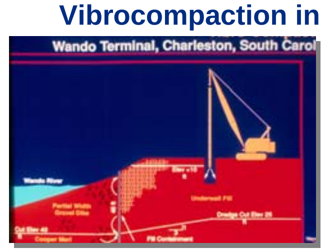


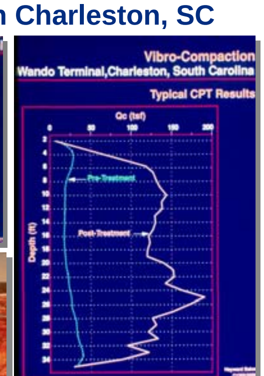
Figure adapted from Hayward Baker, Inc.

Vibrocompaction/Vibroreplacement

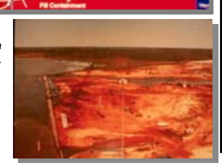
Vibroreplacement

Vibrocompaction in Charleston, SC





Photos adapted from Hayward Baker, Inc.



Deep Dynamic Compaction

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Jet Grouting Systems

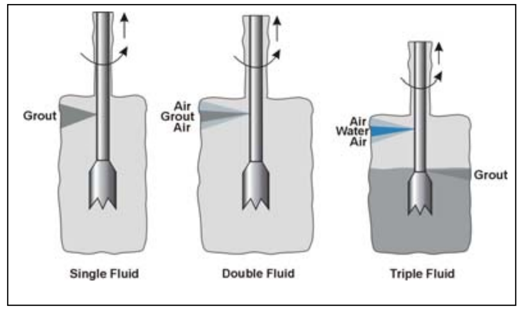

Figure adapted from Hayward Baker, Inc.

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Jet Grouting Process

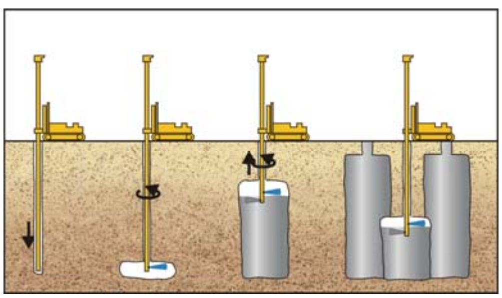

Figure adapted from Hayward Baker, Inc.

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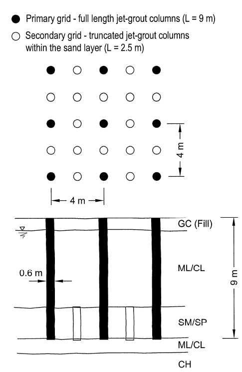
Jet Grouting for Liquefaction Mitigation



FEMA Instructional Material Complementing FEMA 451, *Design Examples* Geotechnical 15-4 - 100

Jet Grouting Machine

Photo courtesy: T. Durgunoglu, Zetas, Inc.

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Geotechnical 15-4 - 101

Excavated Jet-Grout Columns

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Geotechnical 15-4 - 102

Deep Soil Mixing

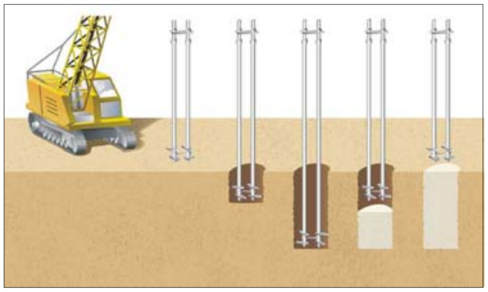


Figure adapted from Hayward Baker, Inc.

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Deep Soil Mixing

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Deep Soil Mixing

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Slopes and Dams

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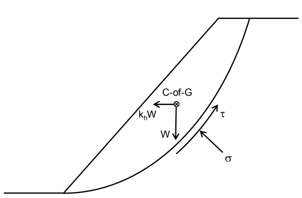
Pseudostatic Analysis



- stability is related to the resisting forces (soil strength) and driving forces (inertial forces)
- seismic coefficient (k_h) to represent horizontal inertia forces from earthquake
- seismic coefficient is related to PGA
- insufficient to represent dynamics of the problem

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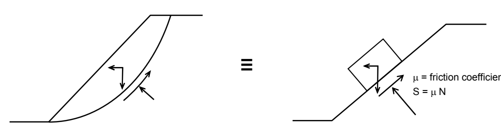
Displacement Analysis



- Estimate the acceleration (i.e. k_h) that would overcome the available friction and start moving the block down the plane – critical acceleration, yield acceleration
- Bracket the acceleration time history with yield acceleration in one direction (i.e. downward movement only), double integrate the portion of the acceleration history to estimate permanent displacement
- Or use simplified charts to relate permanent displacements to yield acceleration and peak ground acceleration

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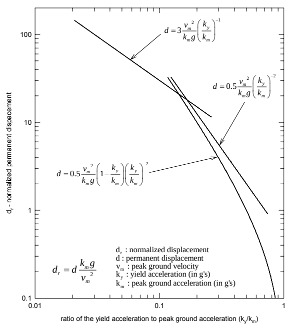
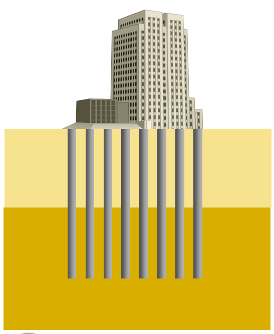
Displacement Analysis



Soil-Structure Foundation Interaction- SSFI

- Traditionally considered conservative to ignore (flexible foundations transmit less motion to superstructure, vice versa);
- However, recent studies from (i.e., 1995 Kobe, Japan EQ) suggest SSFI effects may actually increase ductility demand in some structures

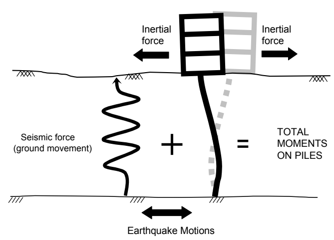
Seismic Design of Pile Foundations - SSFI



- The piles have to withstand forces due to the movement of the soil around and also inertial forces due to the building above

SSFI- Example: Earthquake Loadings on Piles

1. Seismic force;
2. Inertial force;
3. Soil failure (liquefaction, etc.)



Deep Foundations in Soft Soils

IBC 2003 Primary Geotechnical Issues

- Map-based procedure not ideally suited for geotechnical analyses
- Interpretation of soil categories not straight forward (i.e., What is “F” site?)

National Seismic Hazard Maps & IBC Issues for Geotechnical Use

- Maps generalized and not originally intended for site-specific analysis that account for the effects of local soil conditions, such as liquefaction.
- Map-based site classification procedure does not work as well for complex, layered soil profiles (site class based on average of top 30 m or 100 ft.)– think of 30 ft. of medium clay on top of hard rock– should this really be a “C” site?
- Modifications of ground motions for the effects of local soil conditions using the maps is not well-established
- Maps do not account for regional geology

National Seismic Hazard Maps & IBC Issues for Geotechnical Use

- Further away from original design intent, the fewer guidelines are available (structural engineer ⇒ geotech engineer ⇒ seismologist)
- Maps developed mainly for structural design
- Earthquake magnitude/duration not provided directly, only pga’s (M requires deaggregation)
- For structures with elastic response, duration is not as important per se
- Magnitude/duration is very important for most geotechnical analyses (non-linear behavior)

IBC 2003 Geotechnical Design Issues

- Provisions (Chap. 18) recommend $S_{DS}/2.5$ for liquefaction analysis ⇒ SDS factored by 2/3, and 2/3 is from structural considerations, not soil-- this is inconsistent!!
- Structures can factor MCE by 2/3, but not soils ⇒ new IBC Provisions affect geotechnical analyses more than structural analyses
- 20% limitation in reduction of map-based design motions based on site-specific analysis, but no simplified approach available for Class “F” sites ⇒ leads to loophole.
- What is “F” site not always clear (i.e. “liquefaction”)

IBC Geotechnical issues

TABLE 1615.1.2(1)
VALUES OF SITE COEFFICIENT F_a AS A FUNCTION OF SITE CLASS AND MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS (S_S)[a]

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	Note b
F	Note b	Note b	Note b	Note b	Note b

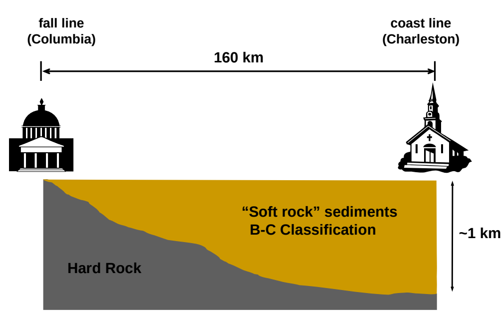
a. Use straight line interpolation for intermediate values of mapped spectral acceleration at short period, S_S.
b. Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values.

TABLE 1615.1.2(2)
VALUES OF SITE COEFFICIENT F_v AS A FUNCTION OF SITE CLASS AND MAPPED SPECTRAL RESPONSE ACCELERATION AT 1 SECOND PERIOD (S_1)[a]

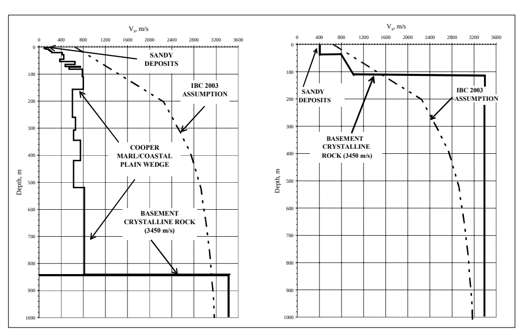
SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT 1 SECOND PERIOD				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	Note b
F	Note b	Note b	Note b	Note b	Note b

a. Use straight line interpolation for intermediate values of mapped spectral acceleration at 1-second period, S_1.
b. Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values.

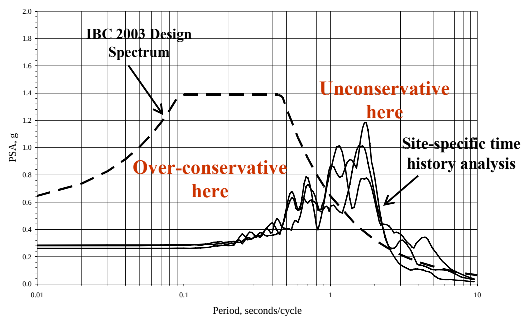
Example of Conditions Different from Those Assumed by Current USGS Maps



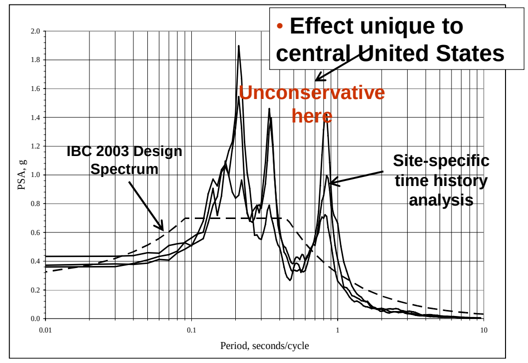
Charleston, SC Columbia, SC



Charleston, SC, Response Spectra

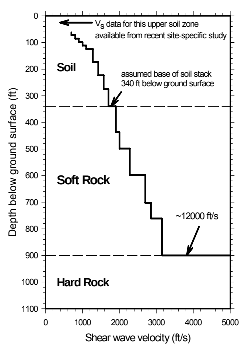


Columbia, SC, Response Spectra -- High Impedance

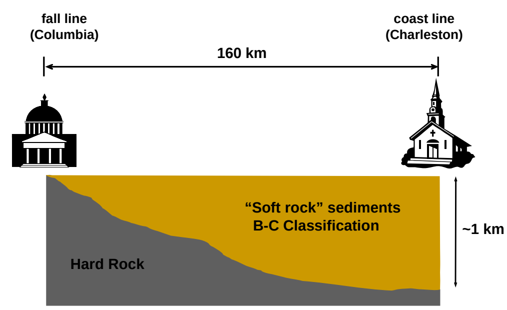


IBC 2003 Site-Specific Example

- Typical South Carolina Coastal Plain Site



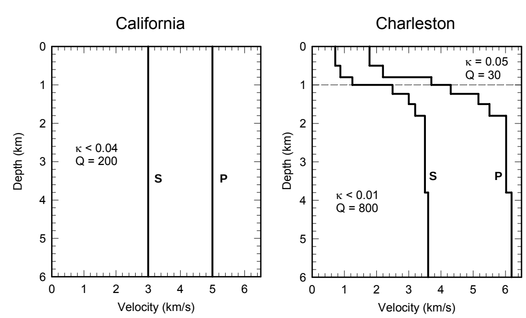
South Carolina Coastal Plain



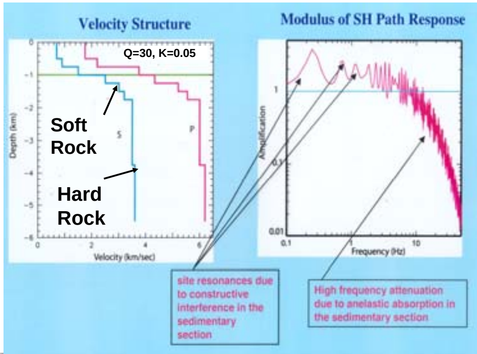
SC Coastal Plain Geology

- SC coastal plain sediments ("soft rock") difficult to characterize
- Q & κ (f of damping) are two big unknowns
- Sediments filter high frequencies and decrease peak motions
- "Effective" κ values in Eastern US soft rock similar to κ values for Western US hard rock
- "Soft rock" motions in coastal SC may be similar to Western US "hard" rock motions

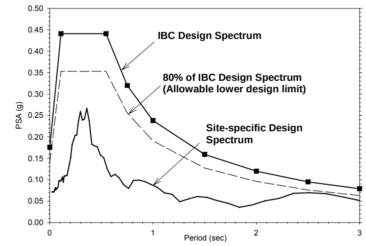
WUS vs. EUS Crustal Models



Effect of SC Coastal Plain on Ground Motions



Results of Site Specific Analysis*

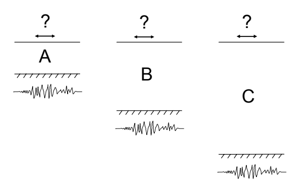


* Includes effect of coastal plain sediments plus near-surface soils in top 30 m. Plots developed for typical site in coastal SC

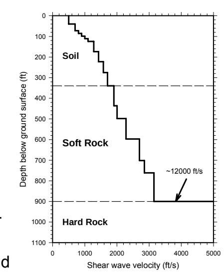
Special Comments on Site Response Analysis in CEUS

- Analysis techniques common in WUS, may not apply in many cases in CEUS
- Site response (i.e., SHAKE) analyses not as straight-forward in CEUS
- SHAKE has depth limitations (600 ft.? CEUS sites can be deeper)
- Where is halfspace? (Vs = 2000 ft/sec rule of thumb not always applicable in CEUS)

Where Is the Halfspace?



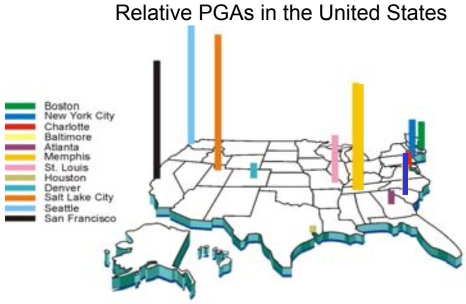
Typical EUS Site:



- Surface motions obtained from A, B, & C would be different, unless base motion modified for the different halfspace depths.
- Deeper profile is probably better to use, if base motion is appropriately developed and if damping is not too high.

A Final Point to Remember….

Relative PGAs in the United States



Soil is the great equalizer:

Summary

- Losses from earthquakes continue to exceed those from other natural hazards (with the exception of megadisasters like Hurricane Katrina).
- Poor soils tend to increase damages from earthquakes.
- Earthquake soil mitigation, especially for soil liquefaction, is effective.

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Summary

- Current IBC 2003 procedures are based on WUS practice and experience.
- IBC provisions may not yet adequately account for unique CEUS conditions.
- Soil conditions in CEUS increase hazard.

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