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.)

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All slides are annotated.

Key Reference

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

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Geotechnical 15-4 - 2

None.

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)

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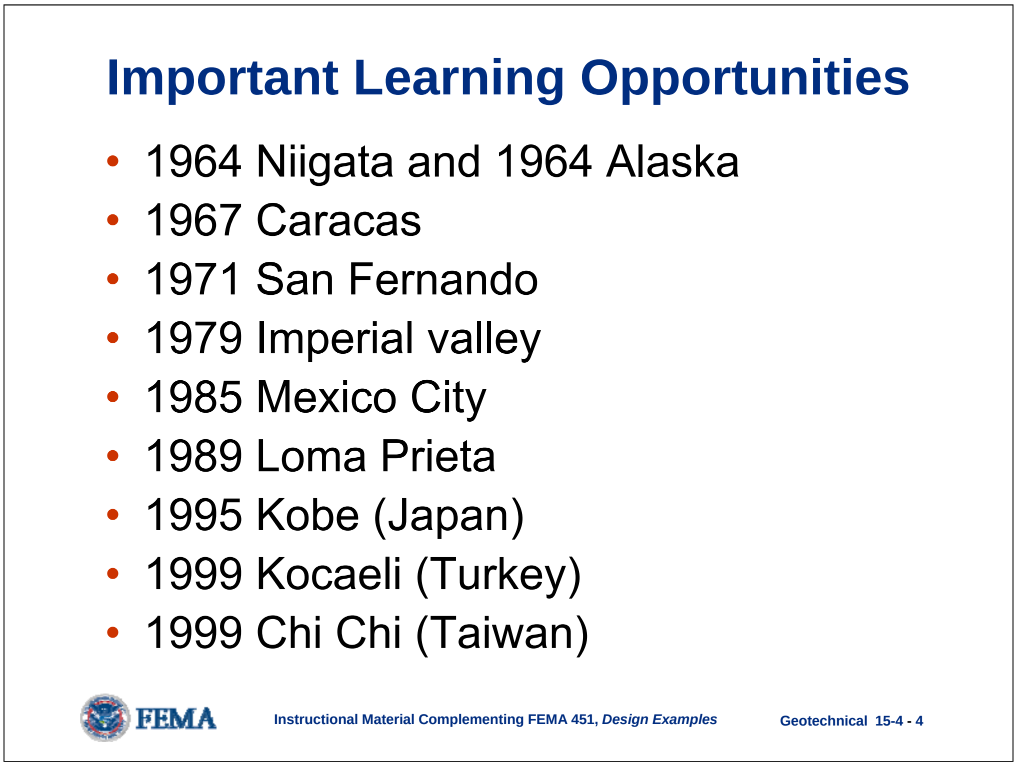
The 1964 Alaska and Niigata, Japan, earthquakes caused the geotechnical engineering community to better appreciate liquefaction and related effects during earthquakes.



These earthquakes each represented important learning opportunities for the geotechnical earthquake engineering community.

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)

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Quotation originally taken from: Rix, G. J., 2001, "Site Response and Ground Motions in the Charleston, SC Area," short-course presentation for NSF-sponsored MAE Center, Mills House Hotel, Charleston, SC, November. Amazing that researchers/observers from this early period noted the importance of site effects.

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

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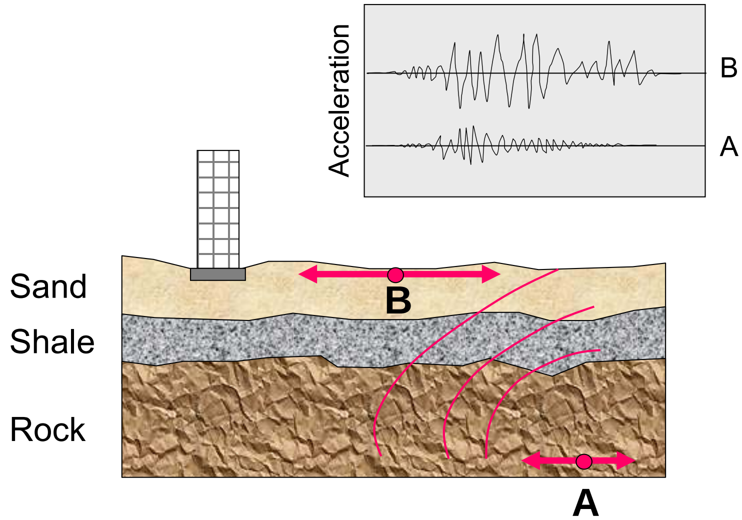
None.

Site Amplification Is Common



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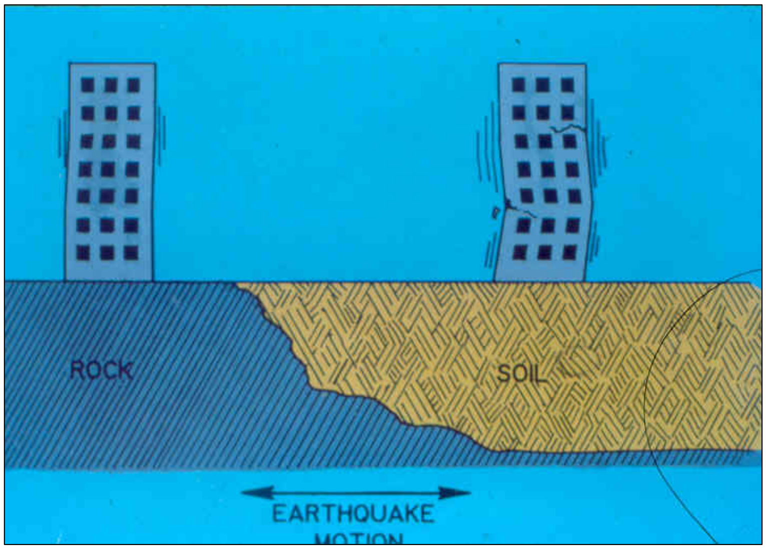
Geotechnical 15-4 - 7

The soil profile acts as filter modifying the amplitude and nature of the motions.

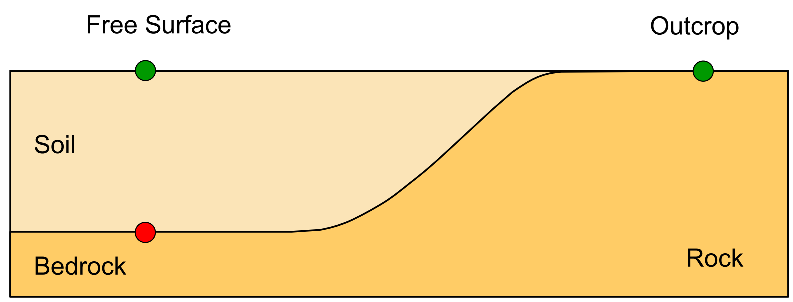
Site Effects on Ground Motions

Conservation of energy drives amplification



Structures founded on soils, especially if soft, tend to be subjected to stronger shaking with longer-period motions. The conservation of energy and the amplification process is important to illustrate here.

Amplification Definitions



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

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

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Amplification can be defined in different ways and the method used should be clearly specified.

Amplification Definitions

- Fourier amplification spectra

$$\left|\frac{a_{\text{free surface}}(f)}{a_{\text{outcrop}}(f)}\right|$$

- Spectral amplification

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

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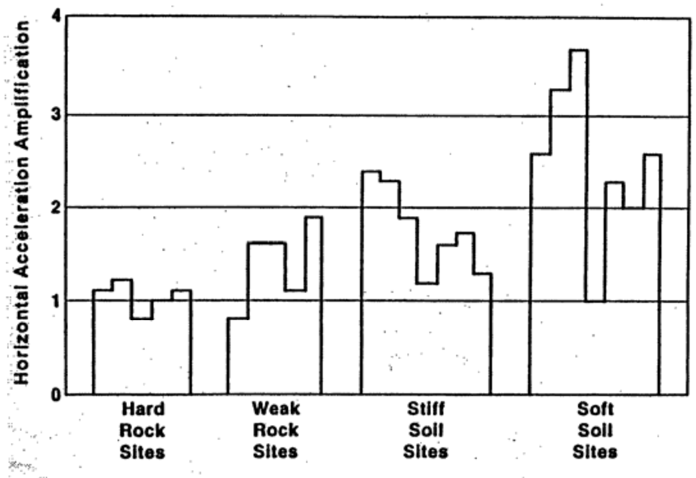
Amplification can be defined differently as depicted in this slide.

Soft Soils Commonly Amplify Motions Relative To Bedrock



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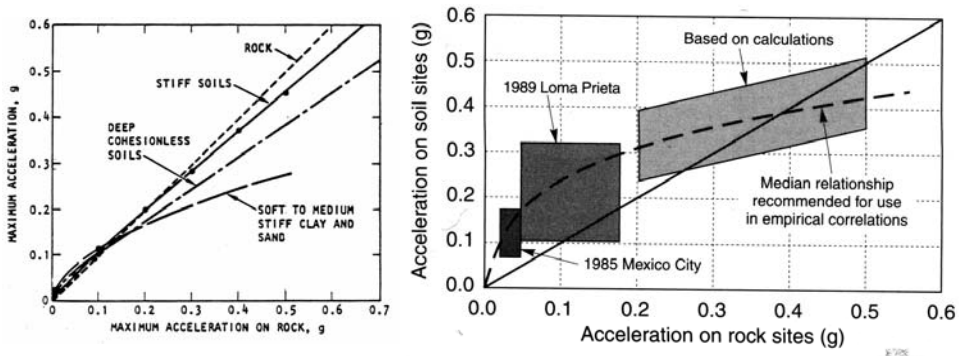
Soils can amplify or de-amplify so codes make conservative assumption that this will occur in specifying general procedures.

Effects of Local Soil Conditions



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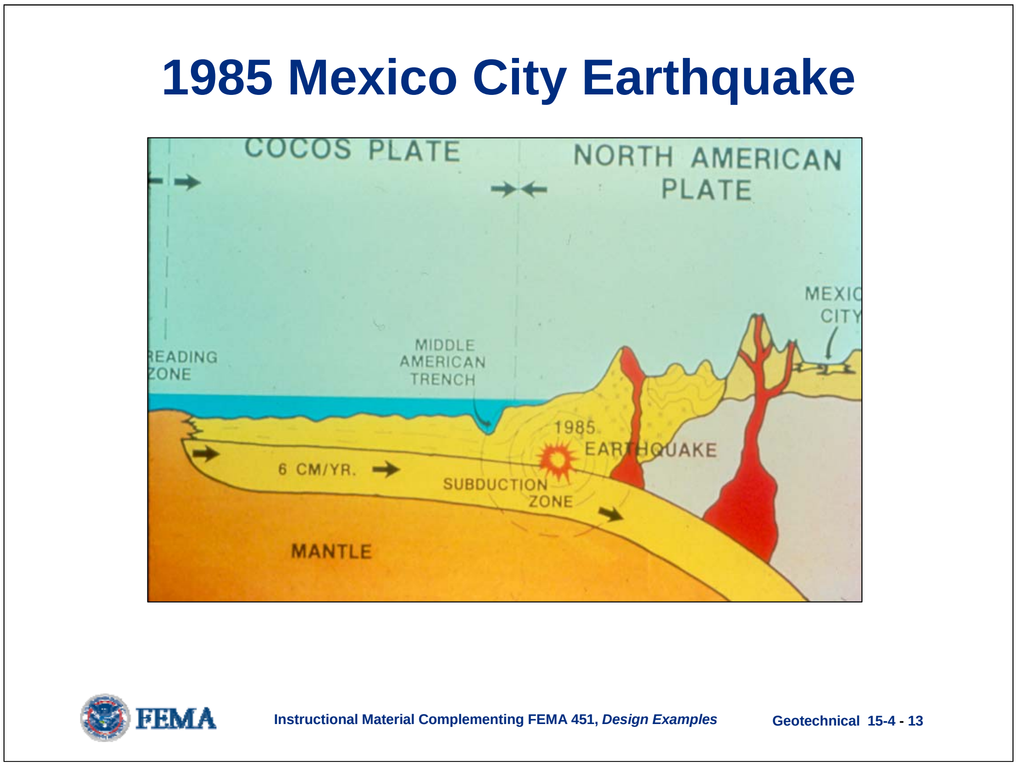
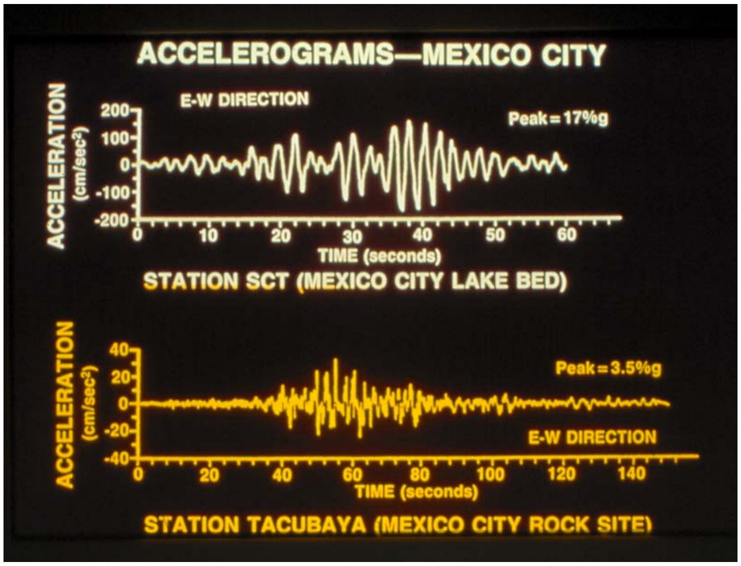
General curves such that shown above are only approximate studies. When such data are used in building codes to account for soil amplification, the most conservative assumption about the amplification potential of the soils is assumed.



The 1985 Mexico City earthquake was centered abut 400 km from the city, but the motions were amplified by the underlying soft sediments (Lake Texcoco).

1985 Mexico City Accelerograms



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Motions on the lake bed were higher and longer in period relative to nearby rock sites.

1985 Mexico City – Juarez Hospital

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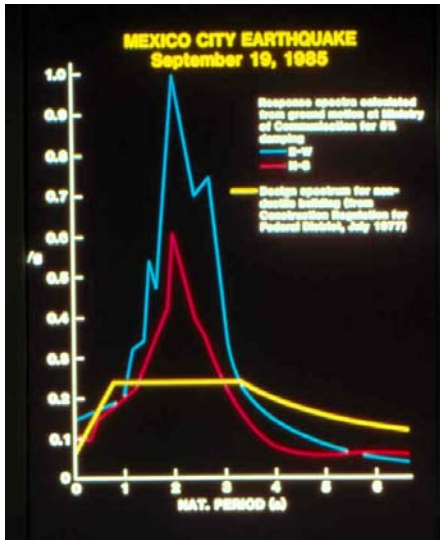
Typical example of tall building failures on lake bed area of Mexico City.

1985 Mexico City – Response Spectra



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Motions on lake bed area of Mexico City (blue curve) were far above the design spectra for that region (yellow curve).

1989 Loma Prieta Earthquake

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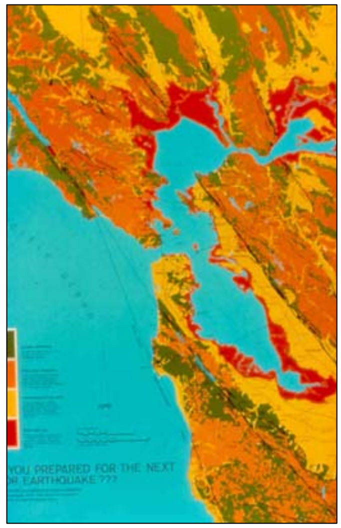
Note the epicenter of the 1989 Loma Prieta earthquake was 65 miles south of San Francisco where most damages occurred.

San Francisco Bay Geological Map

- Soft deposits in red (Bay mud)



Red zones indicate recent soft sediments with a high amplification potential.

San Francisco Marina District

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The Marina District in San Francisco is built atop soft sediments and is susceptible to high amplification (and liquefaction due to sand fill).

Damage in Marina District

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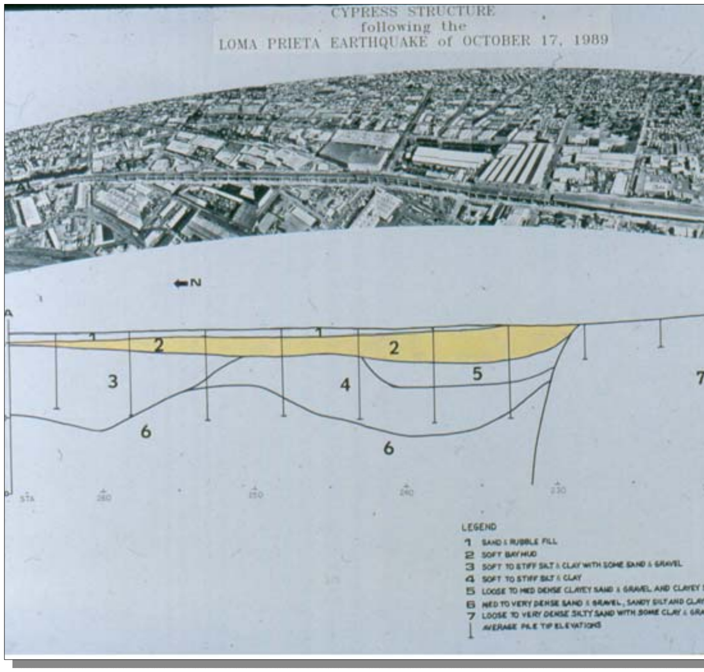
Typical damage in the Marina District following the 1989 earthquake.

Cypress Structure Collapse

Aerial photo of collapsed Cypress Structure. Photo from Idriss (2002).



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Slide from Idriss (2002). Yellow shaded zone indicates soft soils that amplified ground motions above those in nearby stiffer soil areas. The particular section of the Cypress Structure that collapsed was founded on these soft soils.

Cypress Structure Collapse

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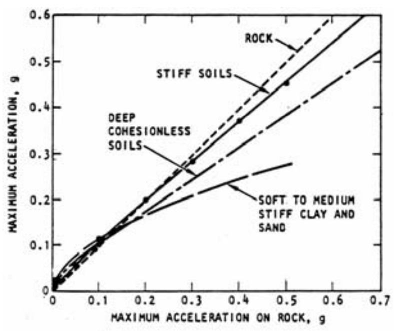
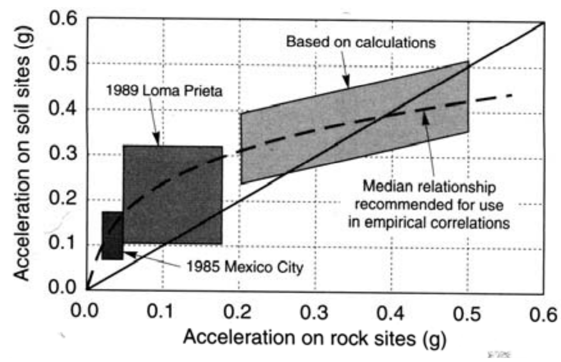
The Cypress Overpass was demolished following the 1989 earthquake.

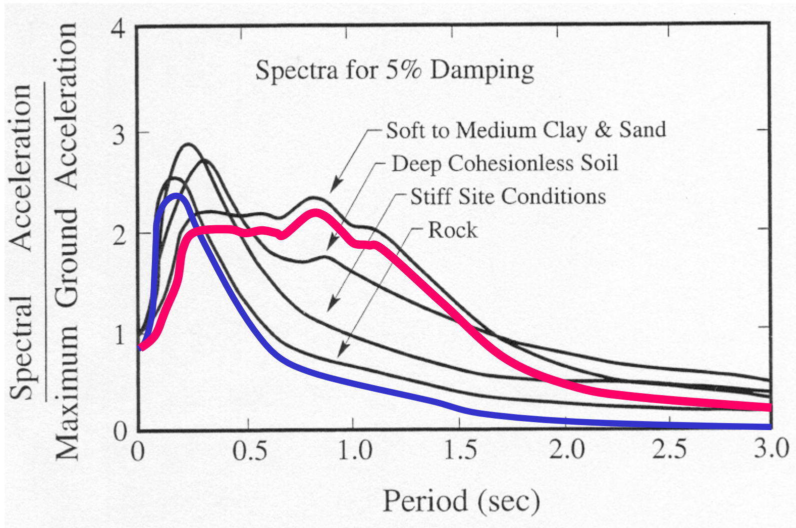
Effects of Local Soil Conditions





General curves such as that shown above are only approximate studies. When such data are used in building codes to account for soil amplification, the most conservative assumption about the amplification potential of the soils is assumed.

Effects of Local Soil Conditions



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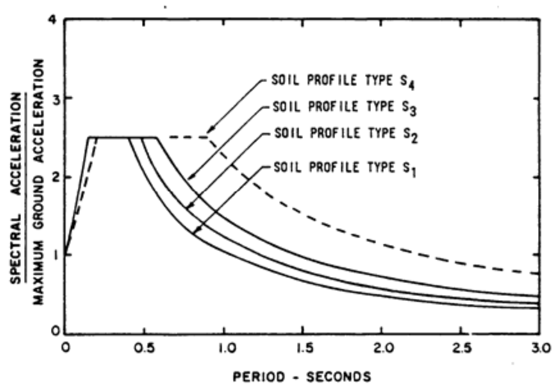
Soft soils decrease the spectral response relative to some stiff soils, but the range over which the motions are near their maximum is broadened.

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

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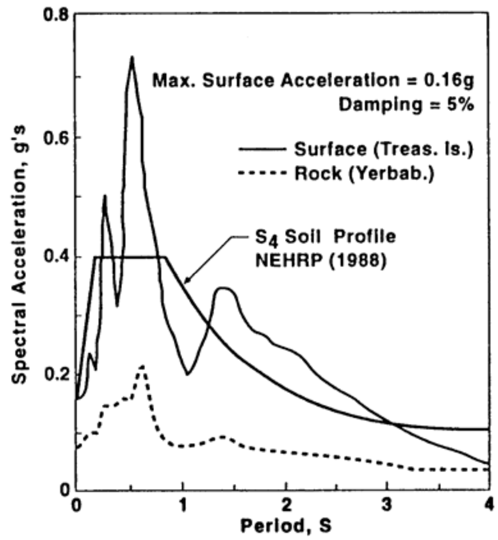
Spectral design curves used prior to the 1989 Loma Prieta earthquake. Note the S4 design curve for soft soils. As will be shown in the following slide, the motions from the 1989 quake greatly exceed the design spectrum being used at the time. The slide was developed from Treasure Island, a deep soft soil site. This prompted the development of Category F for such soils that require site-specific analysis instead of simplified analysis (see following slides from IBC 2003 procedure).

Spectrum from 1989 Loma Prieta at Deep Soft Soil Site



Reason for F Category in IBC 2003

The motions from the 1989 quake greatly exceed the design spectrum being used at the time. The slide was developed from Treasure Island, a deep soft soil site. This prompted the development of Category F for such soils that require site-specific analysis instead of simplified analysis (see following slides from IBC 2003 procedure).

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.

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Note special provisions for F sites. These site conditions are of particular concern for seismic analysis. F site involved soft soils that can greatly amplify ground motions, such as in Mexico City in 1985 or Loma Prieta in 1989 or liquefiable soils.

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	$2{,}500 < \bar{v}_s \leq 5{,}000$	Not applicable	Not applicable
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \leq 2{,}500$	$\bar{N} > 50$	$\bar{s}_u \geq 2{,}000$
D	Stiff soil profile	$600 \leq \bar{v}_s \leq 1{,}200$	$15 \leq \bar{N} \leq 50$	$1{,}000 \leq \bar{s}_u \leq 2{,}000$
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 $w \geq 40\%$, 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 m^2, 1 pound per square foot = 0.0479 kPa.

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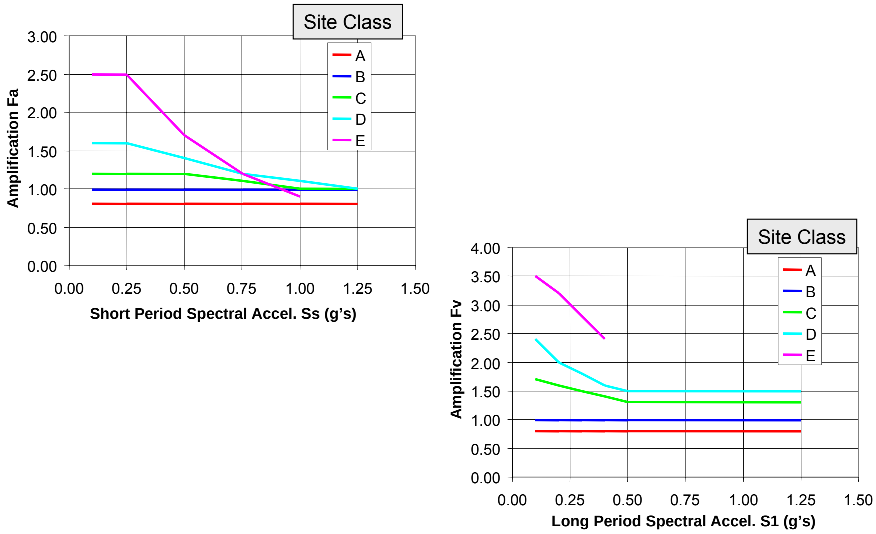
Note special provisions for F sites. These site conditions are of particular concern for seismic analysis. F site involved soft soils that can greatly amplify ground motions, such as in Mexico City in 1985 or Loma Prieta in 1989 or liquefiable soils. Site classification with IBC 2003 is a very important issue. However, this procedure does not work well for highly stratified sites.

NEHRP Provisions Site Amplification for Site Classes A through E



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Note special provisions for F sites. These site conditions are of particular concern for seismic analysis. F sites involved soft soils that can greatly amplify ground motions.

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

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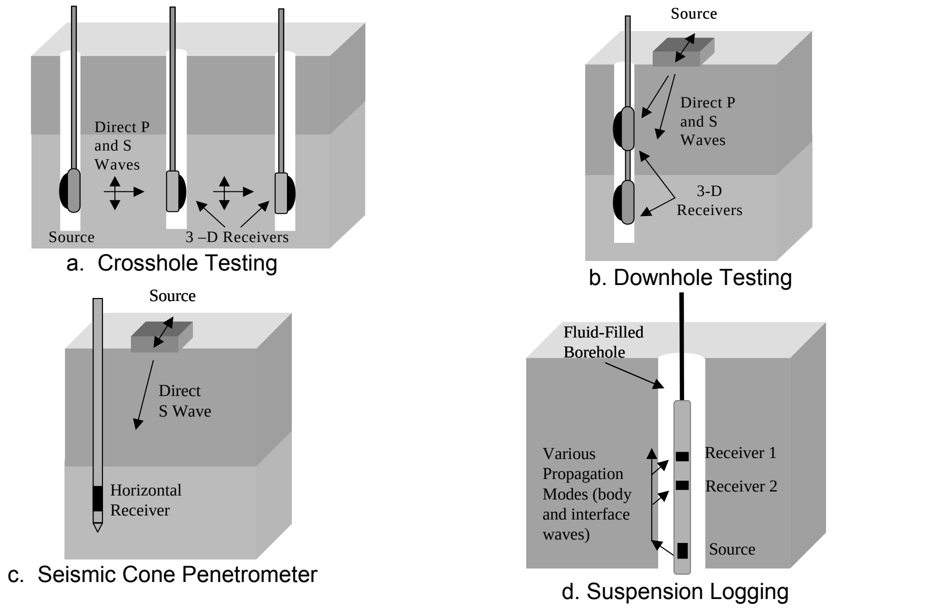
Provisions allow site classification to be determined from various geotechnical data, such as SPT blowcounts, undrained shear strength, and shear wave velocity measurements (V_s); shear wave velocity can be determined economically from CPTs in many cases.

Field Tests To Measure Seismic Wave Velocities



Courtesy of K. H. Stokoe II

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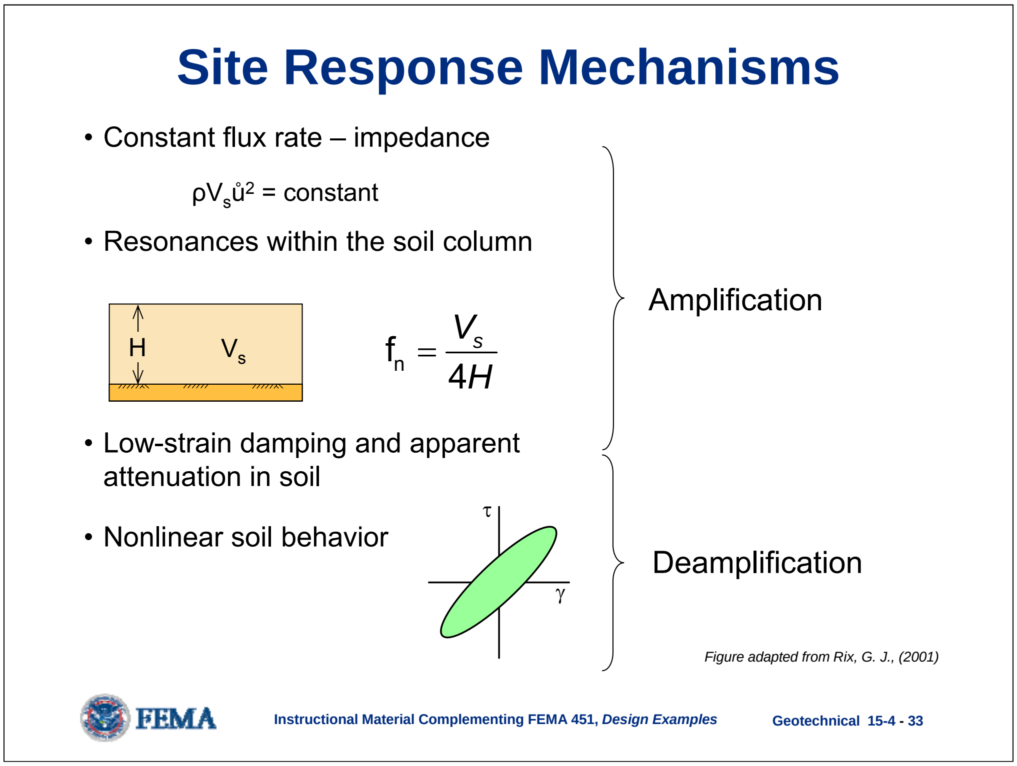
Shear wave velocity can be determined economically from CPTs in many cases; however, cross-hole tests are best.



Some characteristic amplify motions while others cause deamplification. Thus, soil profiles can amplify or deamplify motions relative to what would occur on rock. For building code provision, we conservatively assume amplification will occur on soft soil unless proven otherwise.

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

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None.

(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.

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In modeling soil profiles, it is important to remember that 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)

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There are special issues that should be addressed in cases such as in sediment basins in the central United States (CEUS) where the rock velocities continue to increase with depth for 1 km or more. The assumption of a half space implies no velocity increase below and such the half space must be either placed at great depth or in a location where the effect of this violation of the basic half space assumption is minimized.

Site Response Analysis Techniques

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

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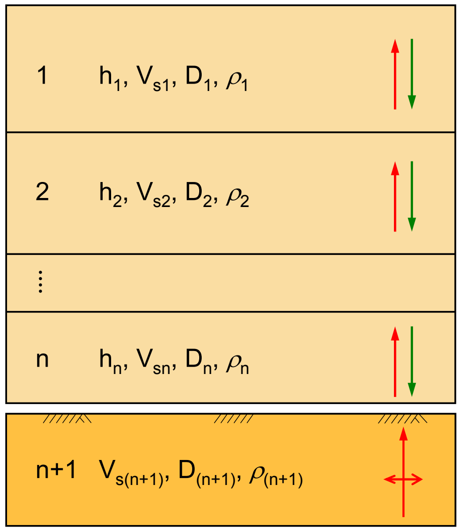
None.

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)

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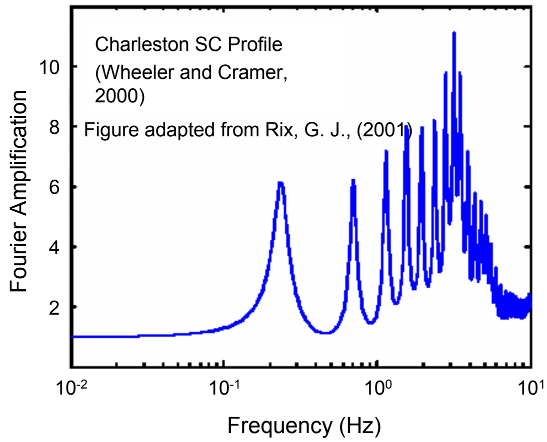
The slide illustrates the basic site response mechanism involved with layer interfaces. Typically, a vertically propagating shear wave is modeled. Site response codes such as SHAKE use this basic approach.

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.

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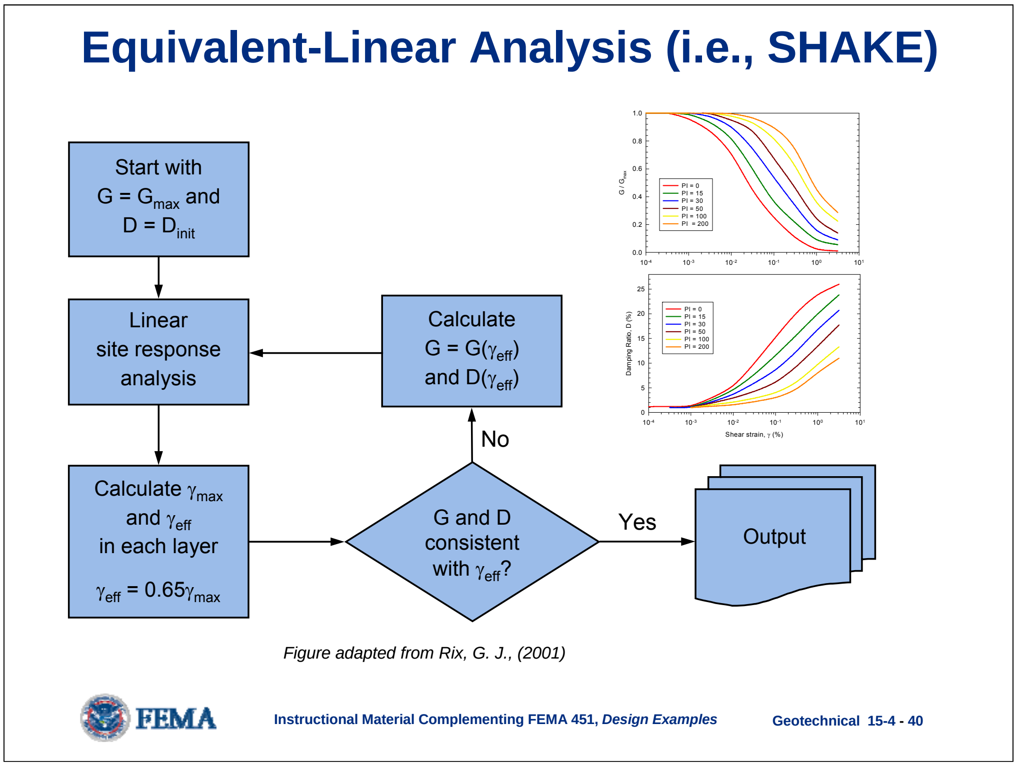
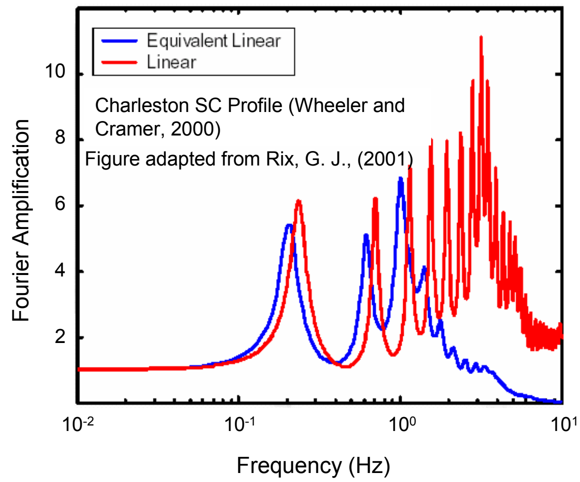
Deep soil unconsolidated sediments beneath Charleston, South Carolina, (about 1 km thick) show amplification of motion in range of 3-5 seconds, the natural period of the sediment stack.



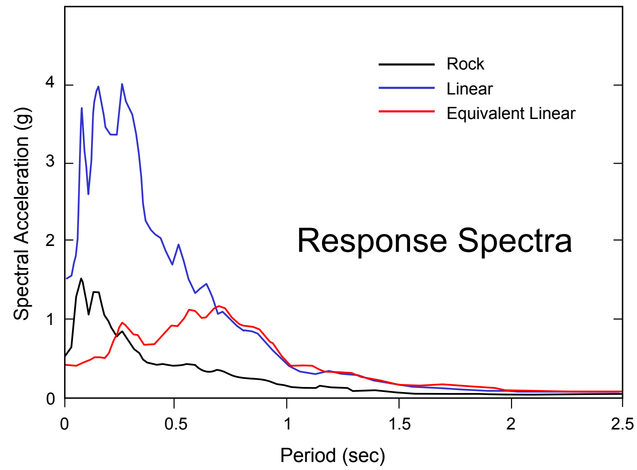
SHAKE uses an iterative procedure to establish strain-compatible dynamic soil properties. The heart of the approach is the degradation curves (dynamic properties vs. strain) used for the analyses.

Equivalent Linear Analysis



The pseudo-nonlinear approach removes a significant amount of high frequency energy relative to the linear approach.

Equivalent Linear Analysis



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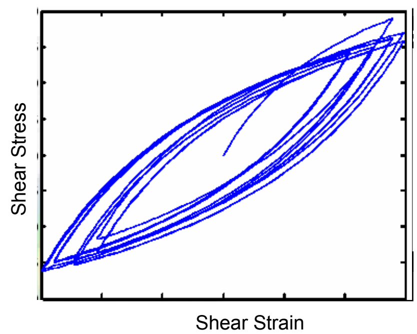
Geotechnical 15-4 - 42

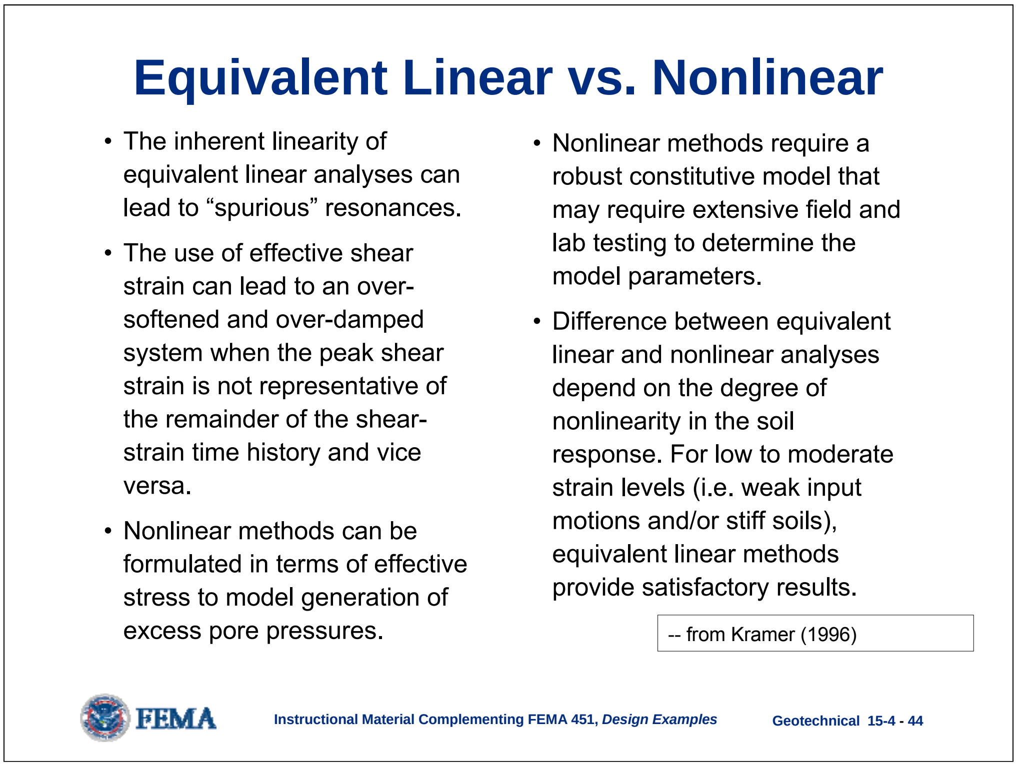
Equivalent linear (SHAKE) motions provide a more realistic approximation of motions (relative to linear methods) for cases where materials do not behave elastically (i.e., soft rock and soils).

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.



Nonlinear codes are typically needed for cases where there is a high degree of strain in the soil or significant pore pressure build up. Such codes can be “twitchy” and unstable.



Adapted 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);

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SHAKE is the most widely used equivalent linear code to predict motions within and atop layered soil profiles.

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)

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Again, nonlinear codes are typically needed for cases where there is a high degree of strain in the soil or significant pore pressure build up. Such codes can be “twitchy” and unstable.

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)

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Two-dimensional codes are used for cases that cannot be modeled in a one-dimensional fashion, such as an earth dam.

Dynamic Soil Properties

- Shear wave velocity profile

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

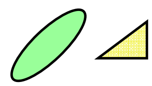
- Nonlinear soil behavior

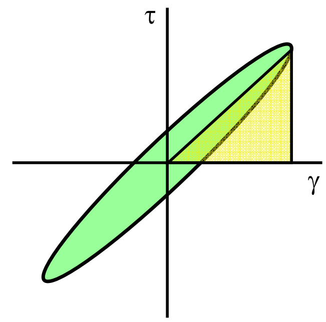
Modulus reduction curve

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

Material damping ratio curve

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





The dynamic soil properties are need for site response analyses. They are either assumed based on experience and local soil condition or are determined by measurement in the laboratory or in situ.

Laboratory Methods

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

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A number of laboratory methods are used to determine dynamic soil properties. Low-strain damping is usually more difficult to determine in the laboratory.

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

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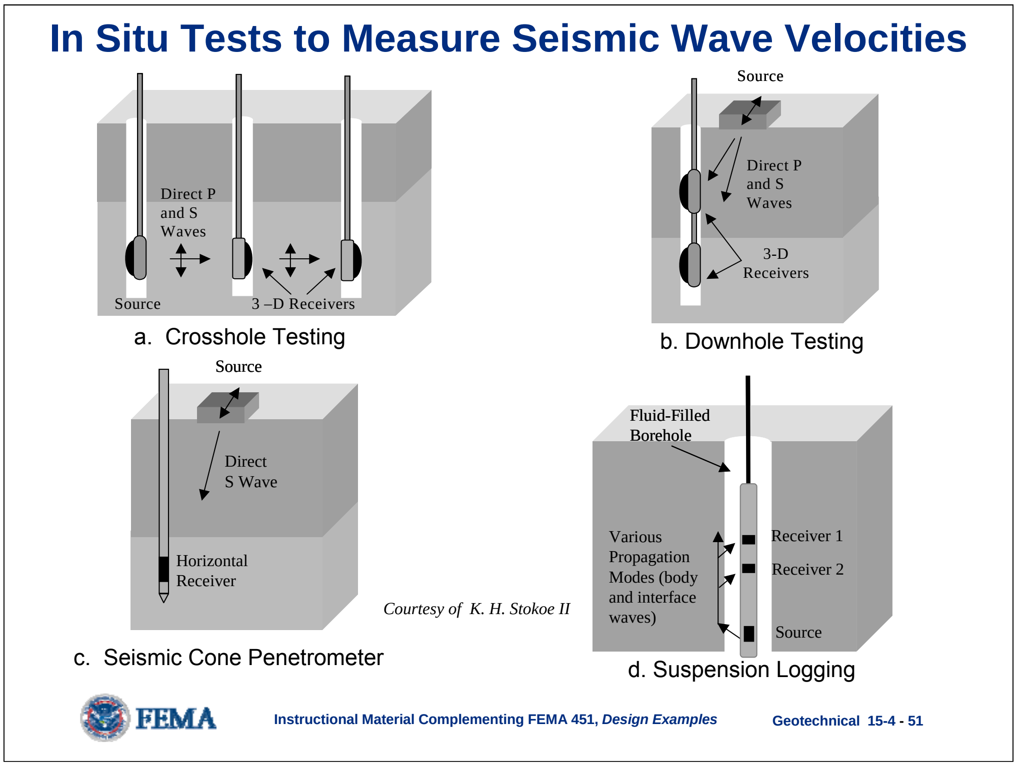
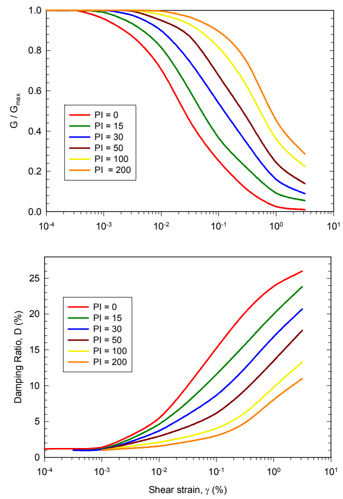
In situ tests are used in lieu of or in conjunction with laboratory methods to determine dynamic soil properties.



Slide from Elgamal (2002). Shear wave velocity can be determined economically from CPTs in many cases; however, cross-hole tests are best.

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)

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The heart of the approach is the degradation curves (dynamic properties vs. strain) used for site response analyses. Such curves are usually determined from careful laboratory tests or from relating various soil parameters (i.e., soil type and plasticity) to published, generalized curves.

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

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There are different definitions for liquefaction depending upon the specific concern. All involve a loss of soil strength and stiffness.

Liquefaction - Field of Sand Boils

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Liquefaction in cabbage field following the 1989 Loma Prieta earthquake. The area shown is the former course of the river shown in the top pf the photo (about 100 years ago according to old geological maps). The clean sands deposited by the river explain the selective pattern of liquefaction in the field.

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This is a close-up shot of the aerial photo in the previous slide.

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This is a photo of a soil boil developing during a Japanese earthquake. Photo from collection of I. Idriss.

Liquefaction Damage, Niigata, Japan, 1964

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Famous photos of buildings on liquefied soils in Niigata, Japan, in 1964 earthquake. The foundation soil beneath this apartment building lost strength during shaking and rotated. Many occupants walked down the sides of the buildings to exit.

Liquefaction Damage, Adapazari, Turkey, 1999

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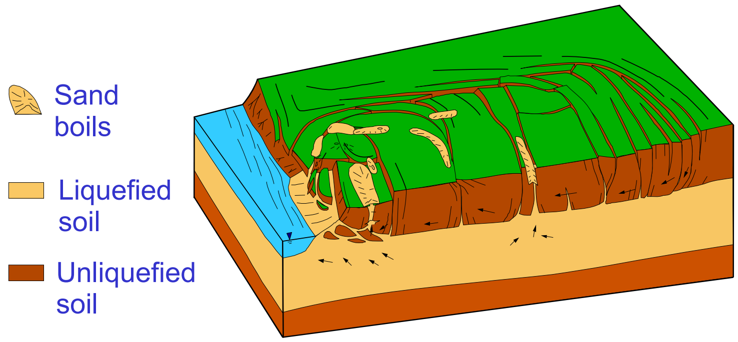
History repeats. The foundation soil beneath this apartment building lost strength during shaking in the 1999 Turkey earthquakes and allowed a foundation bearing capacity failure akin to the famous Niigata photos.

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

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Lateral spreading is the most pervasive form of damage from liquefaction.

Liquefaction and Lateral Spreading, Kobe, Japan, 1995

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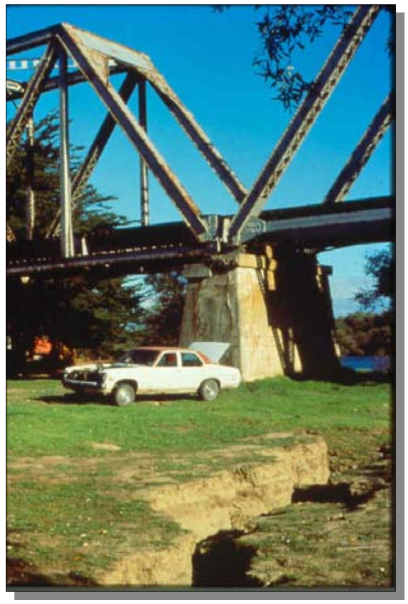
Lateral spreading damage at the Port of Kobe during the M7.3 1995 earthquake. In the 1995 Kobe earthquake, significant damage occurred to port facilities due to liquefaction; after almost 10 years, port trade still is 10 to 15% off.

Lateral Spreading, Loma Prieta, 1989



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Site where bridge pier was shifted due to lateral spreading during the 1989 Loma Prieta earthquake. The railroad track was kinked due to the pier movement and had to be shut down for repair. The bridge was built in 1903 and suffered identical damage during that the 1906 San Francisco earthquake.

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

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

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This slide shows damaged piles beneath a structure found in liquefiable soils subjected to later spreading movement. The soil was excavated and hosed away following the earthquake to reveal the condition of the piles.

Lower San Fernando Dam

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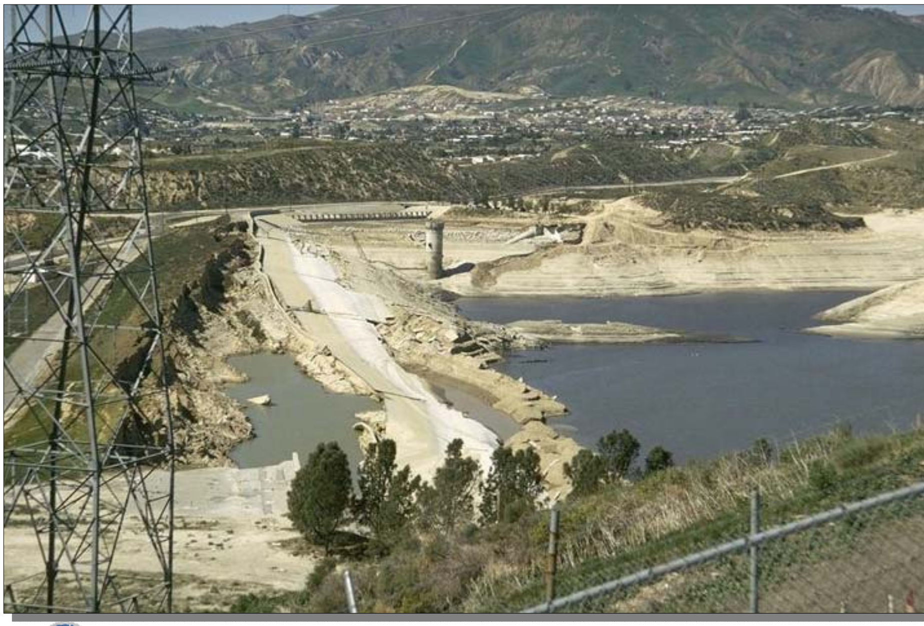
The upstream embankment of the lower San Fernando Dam failed during the 1971 earthquake nearly compromising the reservoir with millions of people downstream.

Lower San Fernando Dam



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The failed upstream embankment of the lower San Fernando Dam can be clearly seen in this photograph with the reservoir drained.

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

None.

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.

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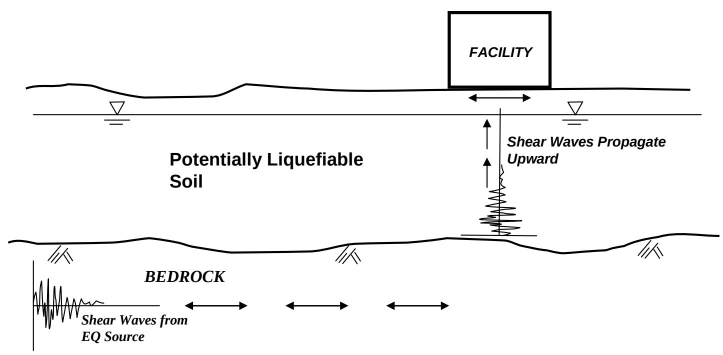
None.

Liquefaction Analysis

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



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Basic steps involved in the liquefaction analysis of these soils are shown on the following pages.

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**).

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The main parameters to be determined in a liquefaction evaluation are the demand and the resistance.

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.

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None.

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)

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None.

Figure 1 – R_d vs. Depth

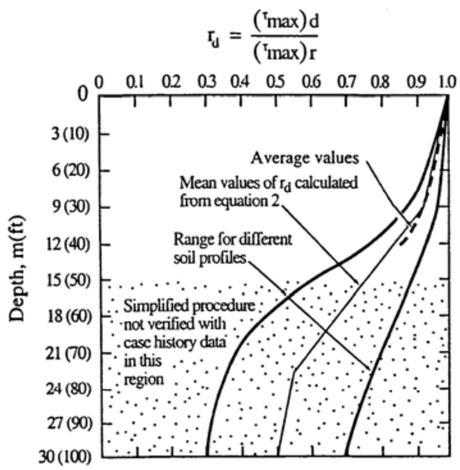

Fig. 1. r_d Versus Depth Curves Developed by Seed and Idriss (1971) with Added Mean Value Lines Plotted from Eq. 2

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None.

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.

None.

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

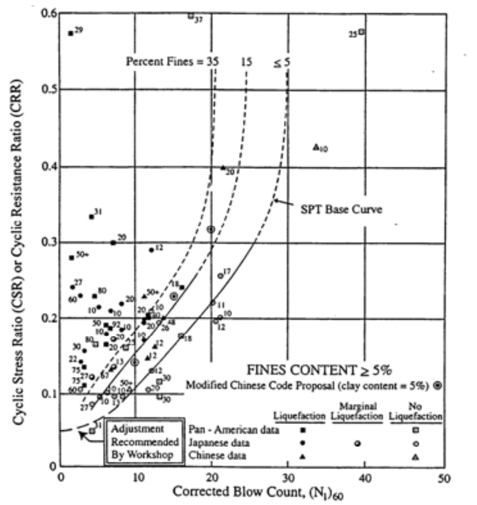

Fig. 2. Base Curve for Magnitude 7.5 Earthquakes Recommended for Calculation of CRR from SPT Data with Data From Compiled Liquefaction Case Histories (modified From Seed et al., 1985)

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Note that the relationship between cyclic resistance and blow count was developed from empirical data (observations).

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)

A number of correction factors are used to standardize blowcount measurements.

Table 2. SPT Correction Factors

Factor	Test Variable	Term	Correction
Overburden Pressure[1]		C_N	$(P_a/\sigma_{vo}')^{0.5}$ $C_N \leq 1.7$
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

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Again, correction factors are used to standardize blowcount measurements.

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).

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None.

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.

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None.

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

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None.

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.

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The factor of safety against liquefaction is determined from the demand and resistance. A factor of at least 1.35 is desired to prevent damages.

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

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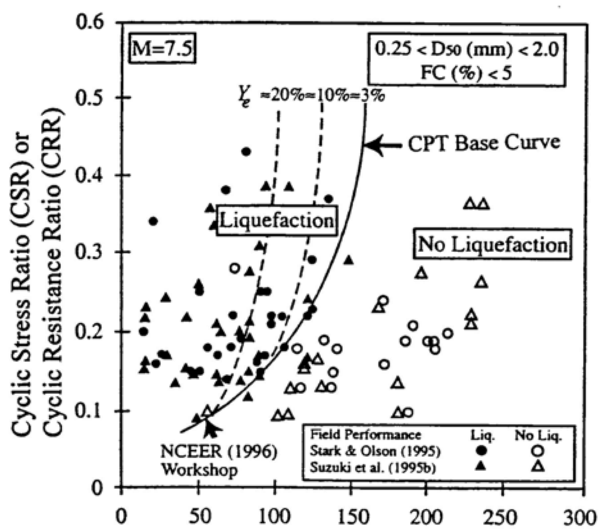
A number of in situ tests have been developed to estimate liquefaction resistance.

Figure 3 - CPT vs. CSR/CRR



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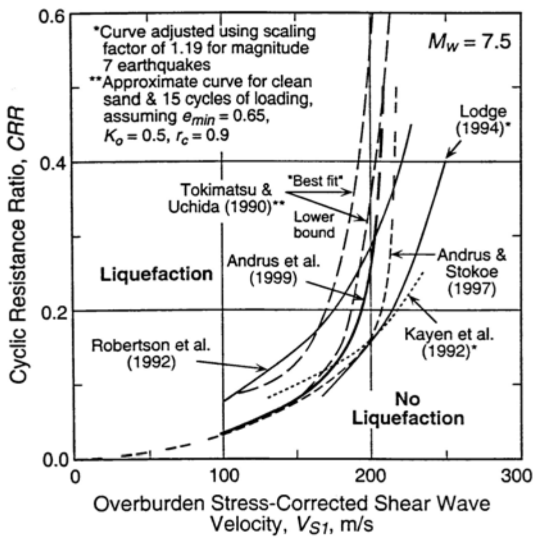
In the past decade or more, CPT curves have been developed similar to those that plot SPT vs. liquefaction resistance.

Figure 4 - Shear Wave Velocity



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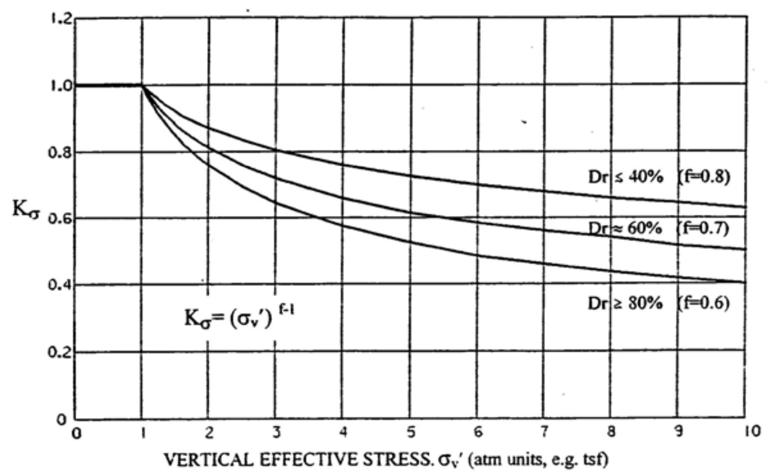
Curves have also been developed for shear wave velocity although strong (tight) correlations and predictions have not yet been achieved.

Figure 5 - Recommended Factors for Kσ



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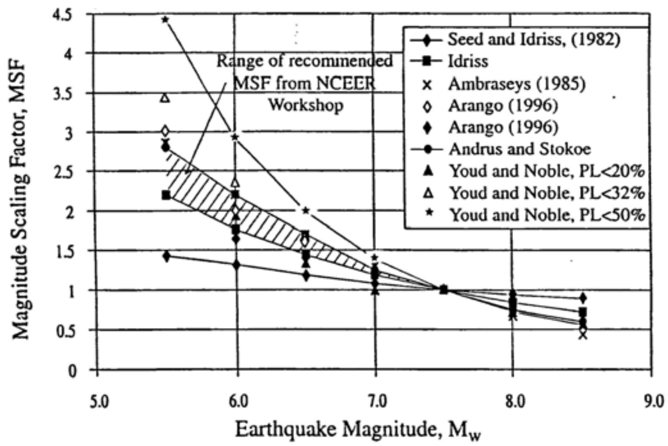
Correction factors are used for cases that depart from level ground near-surface (within about 30 ft.) conditions. The factors shown above, k-sigma, are for conditions with high confining stresses. The correction factors is less than 1.0, as the liquefaction resistance increase with increasing confinement, but not at the same rate as does the confining stress. Thus, the correction factors are less than 1.0 and should be used with the simplified method to predict the resistance of situations with high confinement (deep layers).

Figure 6 - Magnitude Scaling Factors



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Magnitude scaling factors are used to modify the resistance for the standard M7.5 case for other magnitudes. The correction factors are 1.0 for M7.5 and greater than 1.0 for smaller magnitudes (indicating higher resistance with a lower magnitude at the same blowcount). The factors are less than 1.0 for magnitudes greater than M7.5.

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).

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Soils with considerable fines content are difficult to asses and must be specially evaluated. To determine the susceptibility of suspect soils, Chinese criteria are often used – i.e., soil meeting the criteria will liquefy. However, recent work suggests the use of the criterion of “percent finer than 0.005” can be unconservative as soils with much higher clay-sized particle contents higher than 15% can still liquefy (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

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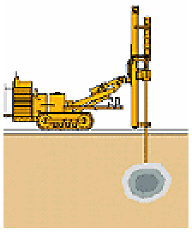
None.

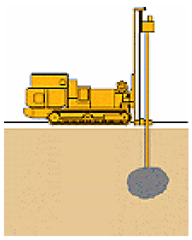
Liquefaction Remediation – Brief Summary

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.

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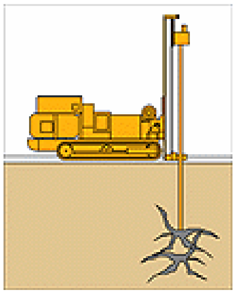
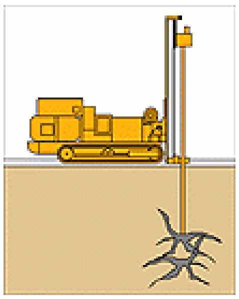
None.



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.

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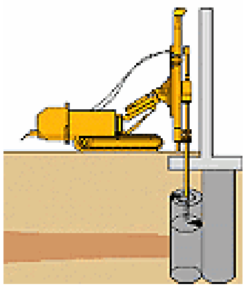
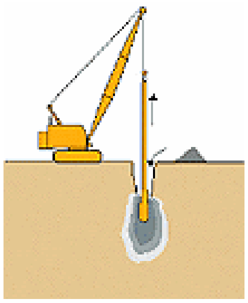
None.



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.

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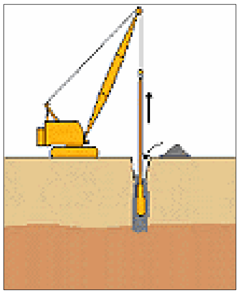
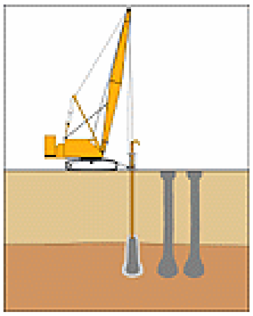
None.



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.

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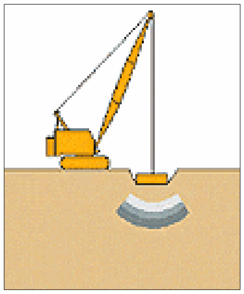
None.



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.

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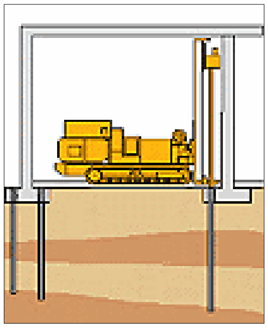
None.

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/

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None.

Vibrocompaction/Vibroreplacement

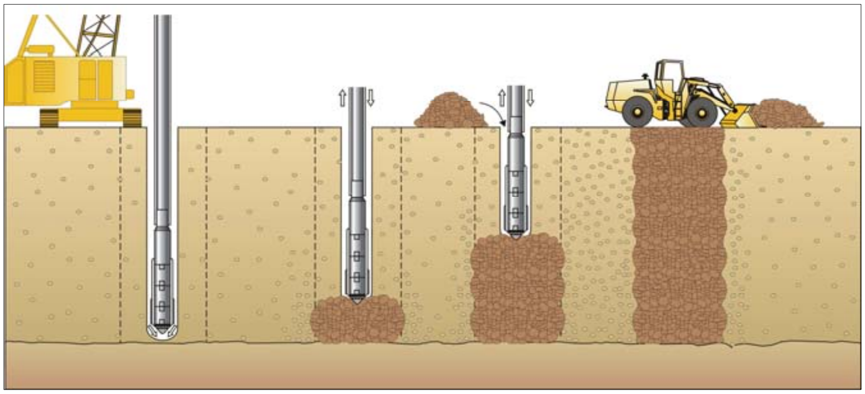

Figure adapted from
Hayward Baker, Inc.

Vibrocompaction/virboreplacement can be an economical liquefaction mitigation approach depending largely upon the fines content of the soil and depth to firm bearing stratum.

Vibrocompaction/Vibroreplacement

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The process of vibrocompaction causes densification of the ground and voids that have to be continuously filled at the ground surface. Consumption rates are typically 1 cubic yard per minute.

Vibroreplacement

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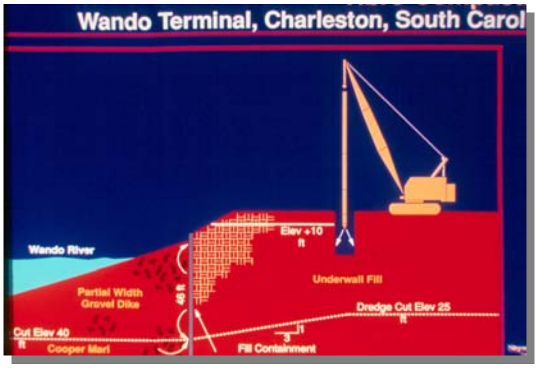
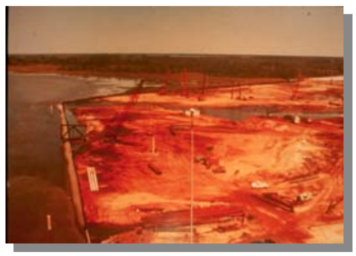
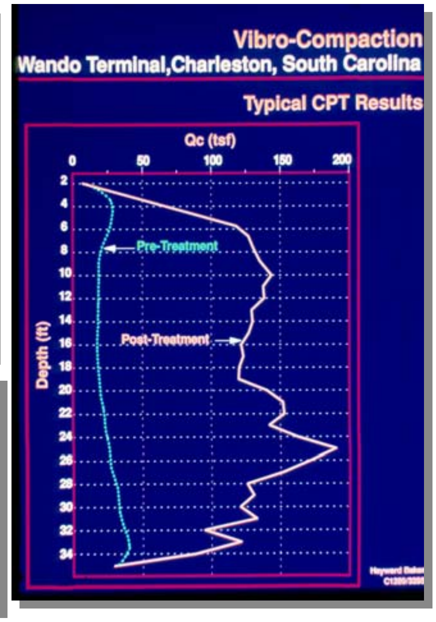
Vibroreplacement involves significant quantities of stone and/or high quality fill being added and densified.

Vibrocompaction in Charleston, SC







Photos adapted from Hayward Baker, Inc.

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The Wando Terminal in Charleston, South Carolina, was treated using vibrocompaction to mitigate the effects of anticipated large earthquake motions that would cause liquefaction.

Deep Dynamic Compaction

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DDC or heavy tamping can be used as an economical alternative in many areas where neighboring structures are not too close (to avoid vibration damage) and where the groundwater table is at least 5 ft below the surface.

Jet Grouting Systems

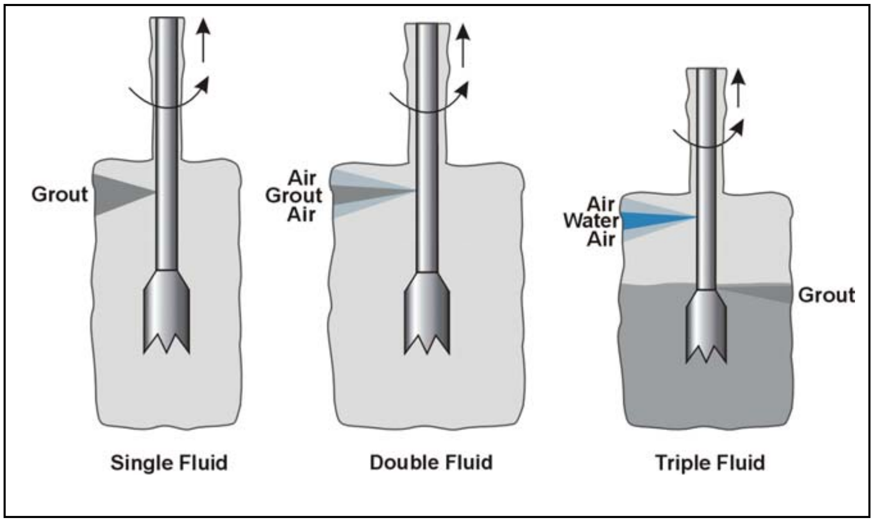

Figure adapted from Hayward Baker, Inc.

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Grout is pumped through the rod and exits the horizontal nozzle(s) in the monitor at high velocity [approximately 650 ft/sec (200m/sec)]. This energy breaks down the soil matrix and replaces it with a mixture of grout slurry and in situ soil (soilcrete). Single fluid jet grouting is most effective in cohesionless soils.

Double Fluid Jet Grouting (Soilcrete D)

A two phase internal fluid system is employed for the separate supply of grout and air down to different, concentric nozzles. The grout erodes in the same effect and for the same purpose as with Single Fluid. Erosion efficiency is increased by shrouding the grout jet with air. Soilcrete columns with diameters over 3 ft can be achieved in medium to dense soils and more than 6 ft in loose soils. The double fluid system is more effective in cohesive soils than the single fluid system.

Triple Fluid Jet Grouting (Soilcrete T)

Grout, air, and water are pumped through different lines to the monitor. Coaxial air and high-velocity water form the erosion medium. Grout emerges at a lower velocity from separate nozzle(s) below the erosion jet(s). This separates the erosion process from the grouting process and tends to yield a higher quality soilcrete. Triple fluid jet grouting is the most effective system for cohesive soils.

Jet Grouting Process

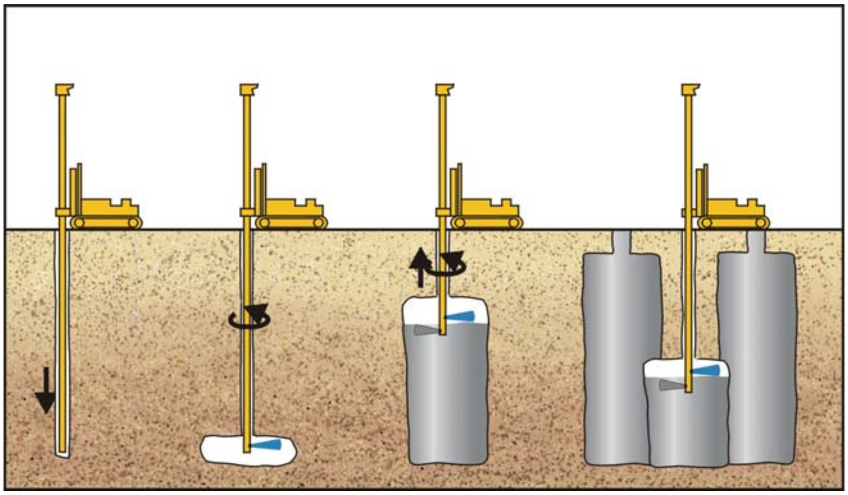

Figure adapted from Hayward Baker, Inc.

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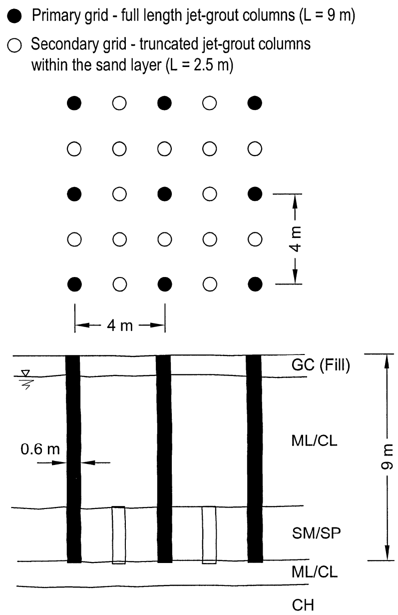
The jet grouting process is performed in distinct stages as shown above for a two-fluid system.

Jet Grouting for Liquefaction Mitigation



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Jet grouting (using discrete columns) has been shown to work well in mitigating liquefaction-related damages; see Martin, J.R., Olgun, C.G., Mitchell, J.K., and Durgunoglu, H.T. (2004), “High-Modulus Columns for Liquefaction Mitigation,” *Journal of Geotechnical* and Geoenvironmental Engineering, ASCE, Vol. 130, No. 6, June 2004, pp. 561-571.

Jet Grouting Machine

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Here the two-nozzle jet grout rig is being tested before insertion into the ground at this site in Turkey. Photo courtesy of T. Durgunoglu, Zetas, Inc.

Excavated Jet-Grout Columns

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Note variability of column diameters. Strengths also vary greatly. Uncertainty of column diameters is a major source of inefficiency in jet grouting, and one reason this technology is relatively expensive.

Deep Soil Mixing

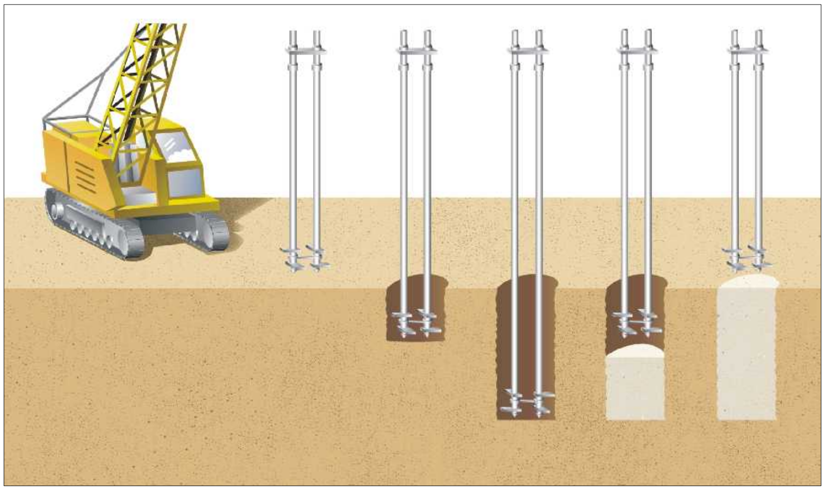

Figure adapted from Hayward Baker, Inc.

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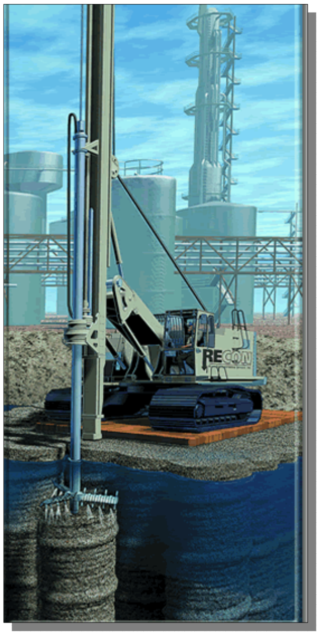
Soil mixing is an in situ soil treatment and improvement technology mechanically blending the in situ soil with cementitious materials that are referred to as binders using a hollow stem auger and paddle arrangement. The intent of the soil mixing method is to achieve improved soil properties. The cemented material that is produced generally has a higher strength, lower permeability, and lower compressibility than the native ground although total unit weight may be less. The properties obtained reflect the characteristics of the native soil, the mixing method, and the binder characteristics.

Deep Soil Mixing



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None.

Deep Soil Mixing

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Large rigs are need to perform soil mixing operations. Mobilization costs are typically high for these machines.

Slopes and Dams

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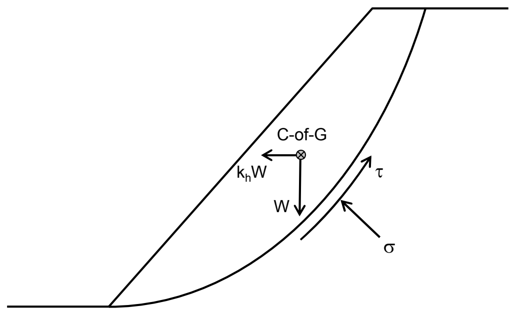
Landslides, including debris avalanches from volcanoes, have been caused by earthquakes. Earthquake-induced acceleration can produce additional downslope force causing otherwise stable or marginally stable slopes to fail. In the 1964 Alaska earthquake, for instance, most rockfalls and debris avalanches were associated with bedding plane failures in bedrock, probably triggered by this mechanism. In addition, liquefaction of sand lenses or changes in pore pressure in sediments trigger many coastal bluff slides.

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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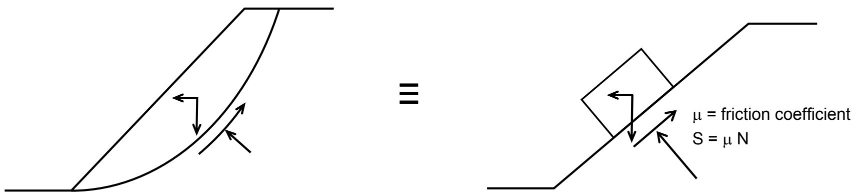
Pseudostatic analysis is used for simple slope design to account for seismic forces.

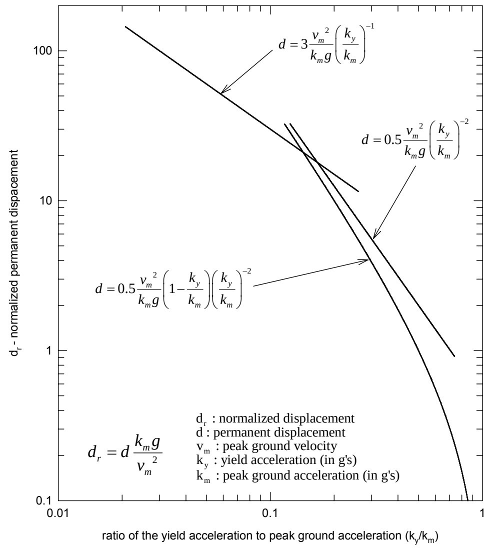
Displacement Analysis



- 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

Displacement analysis is used to estimate the amount of permanent displacement suffered by a slope due to strong ground shaking.

Displacement Analysis



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Simplified charts are developed for displacement analyses to estimate the amount of permanent displacement suffered by a slope due to strong ground shaking.

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

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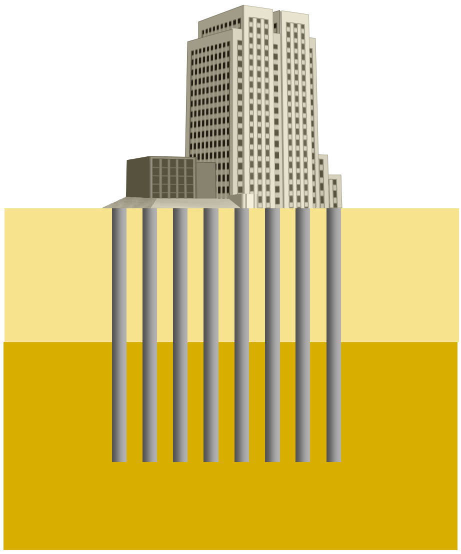
None.

Seismic Design of Pile Foundations - SSFI

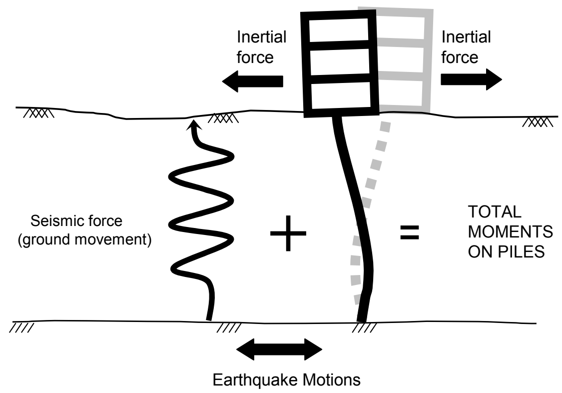


- The piles have to withstand forces due to the movement of the soil around and also inertial forces due to the building above

Soil structure interaction considers the effect of the foundation upon the soil and vice versa and the combined effects upon the superstructure.

SSFI- Example: Earthquake Loadings on Piles

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



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The forces on the piles are from the soil movement due to the earthquake motions as well as those from the attached structure above which is also inducing moments in the piles.

Deep Foundations in Soft Soils

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The response of piles in soft soils continues to be an important consideration and difficult problem to fully understand.

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?)

There are a number of specific IBC 2003 geotechnical issues and concerns.

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

None.

National Seismic Hazard Maps & IBC Issues for Geotechnical Use

- Further away from original design intent, the fewer guidelines are available (structural engineer $\Rightarrow$ geotech engineer $\Rightarrow$ seismologist)
- Maps developed mainly for <u>structural</u> 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 <u>very</u> important for most geotechnical analyses (non-linear behavior)

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None.

IBC 2003 Geotechnical Design Issues

- Provisions (Chap. 18) recommend $S_{DS}/2.5$ for liquefaction analysis $\Rightarrow$ 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 $\Rightarrow$ 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 $\Rightarrow$ leads to loophole.
- What is "F" site not always clear (i.e. "liquefaction")

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None.

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.

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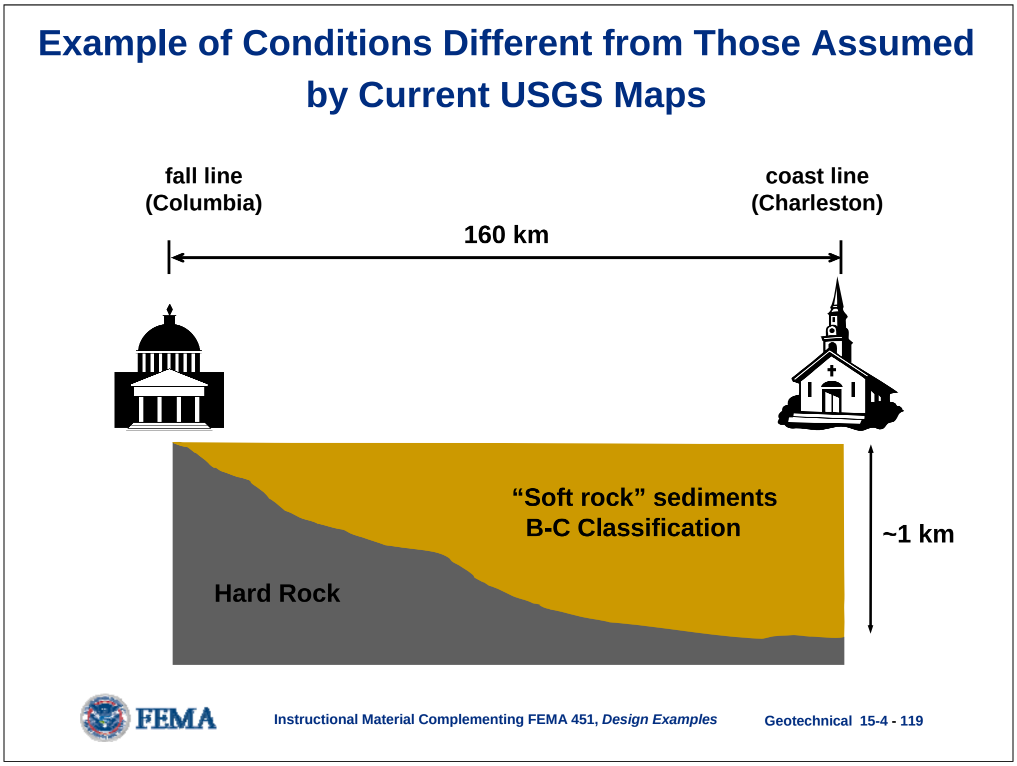
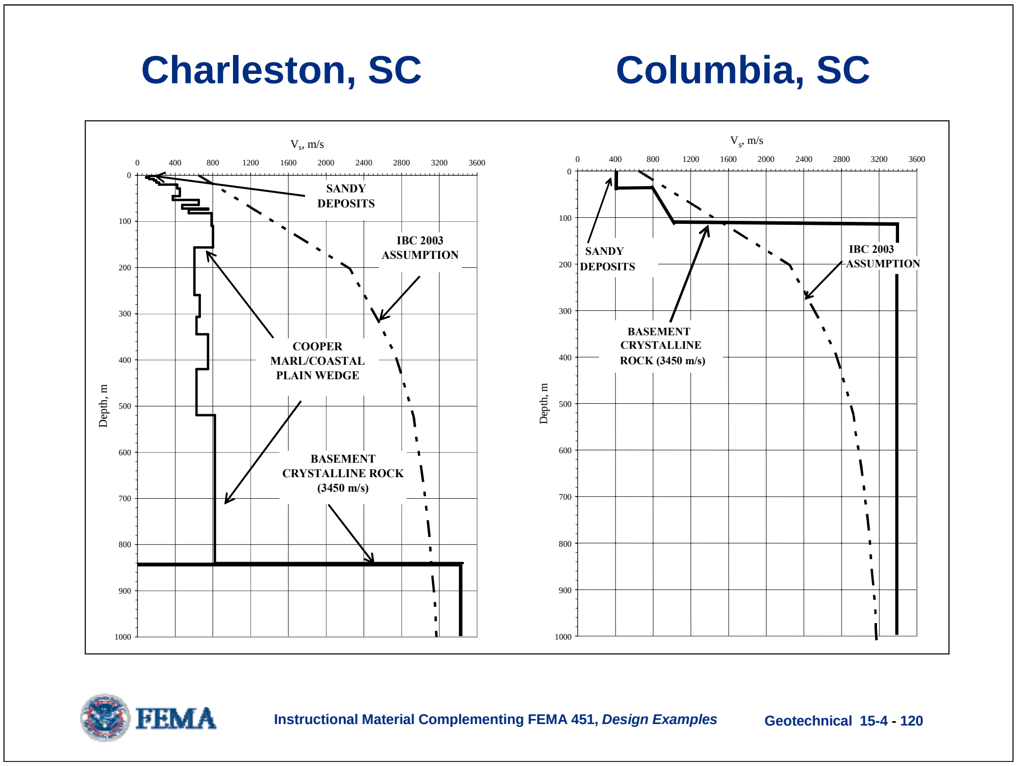
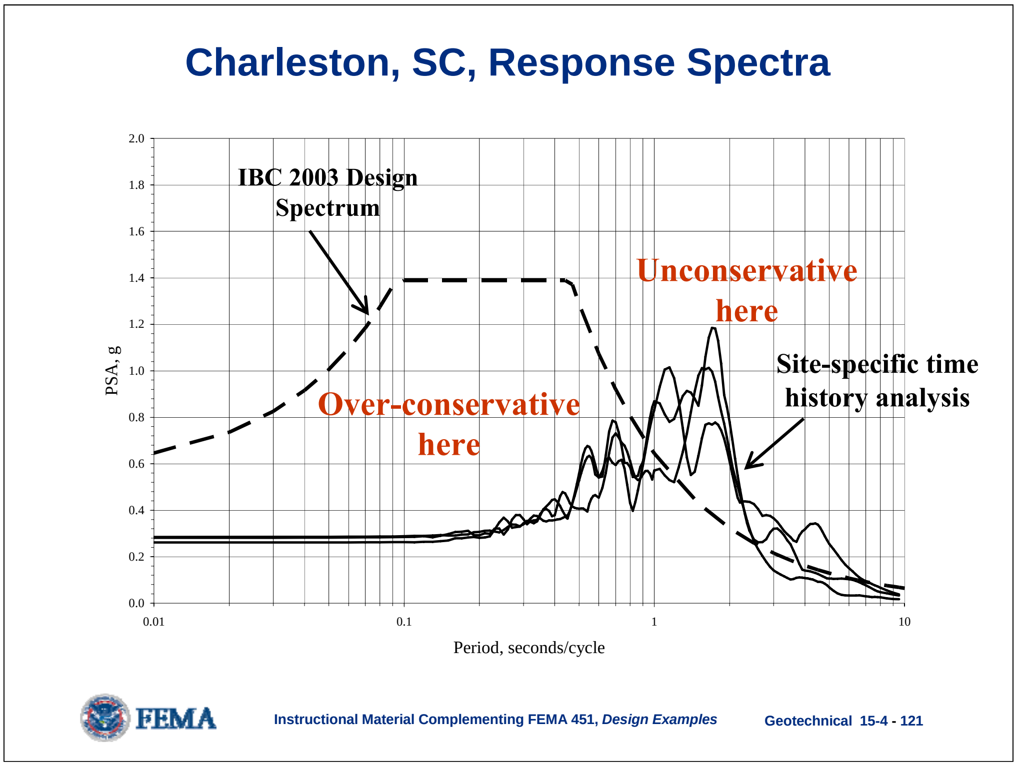
Again, the F sites are of primary concern and require site-specific analysis.



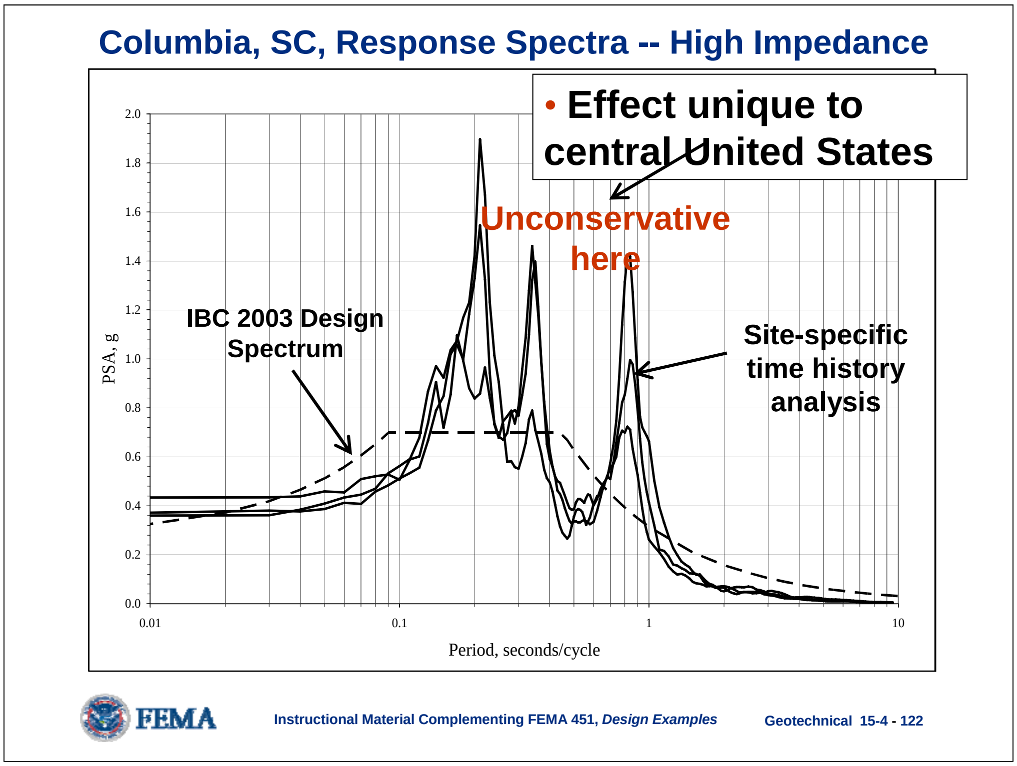
The geologic conditions in the coastal plain of South Carolina are an excellent example of conditions that are different from those assumed during the development of the USGS maps. Specific differences are shown on the following slides.


Charleston, SC
Columbia, SC
Vs, m/s
Depth, m
SANDY DEPOSITS
IBC 2003 ASSUMPTION
COOPER MARL/COASTAL PLAIN WEDGE
BASEMENT CRYSTALLINE ROCK (3450 m/s)
BASEMENT CRYSTALLINE ROCK (3450 m/s)
FEMA
Instructional Material Complementing FEMA 451, Design Examples
Geotechnical 15-4 - 120

None.



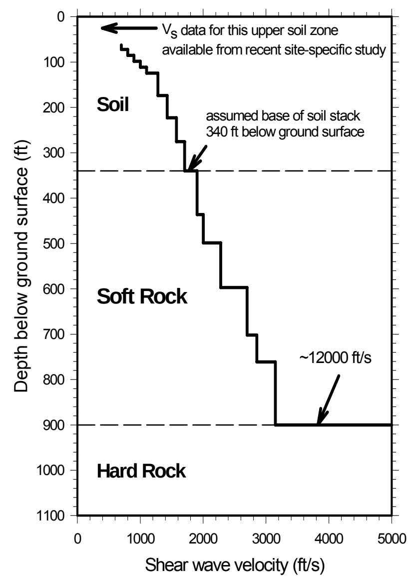
The effects of the soil conditions beneath Charleston, a deep wedge of sediments nearly 1 km thick, is shown. The peak and low-period motions are damped whereas the longer period low frequency motions are amplified.



The effects of the soil conditions beneath Columbia, an area where hard rock is close to the ground surface is shown. The peak and low-period motions are greatly amplified relative to the IBC 2003 building code design spectrum.

IBC 2003 Site-Specific Example

- **Typical South Carolina Coastal Plain Site**



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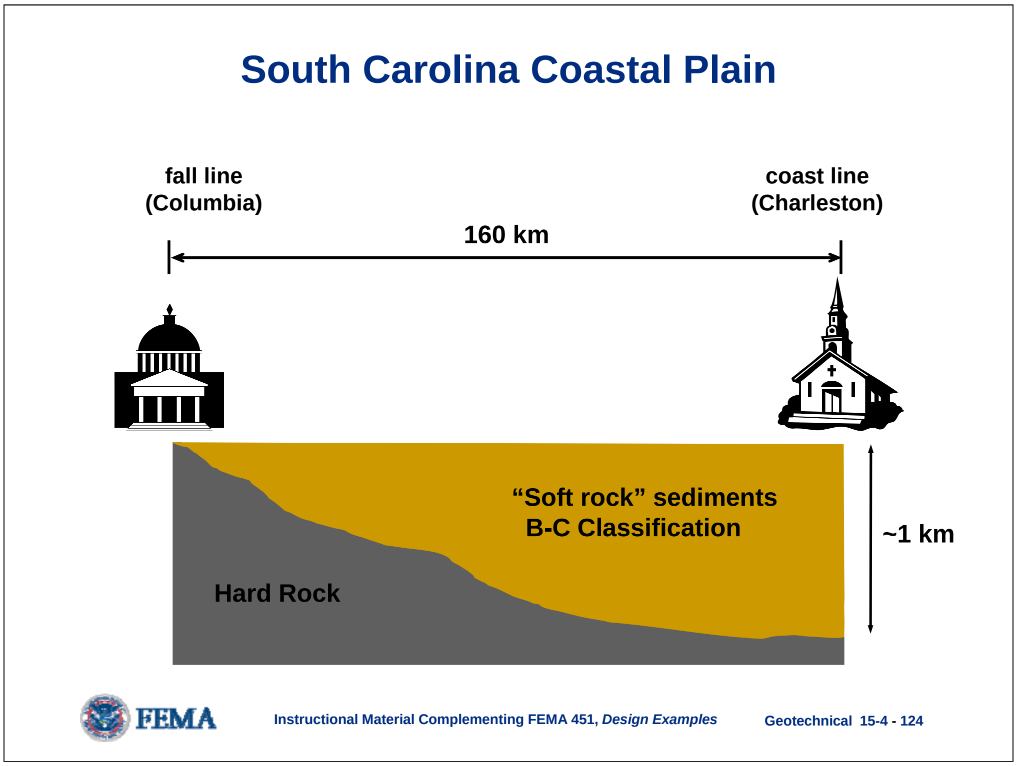
None.



The thick wedge of coastal plain sediments greatly affects earthquake motions. Again, the geologic conditions in the coastal plain of South Carolina are an excellent example of conditions that are different from those assumed during the development of the USGS maps. Specific differences are shown on the following slides.

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

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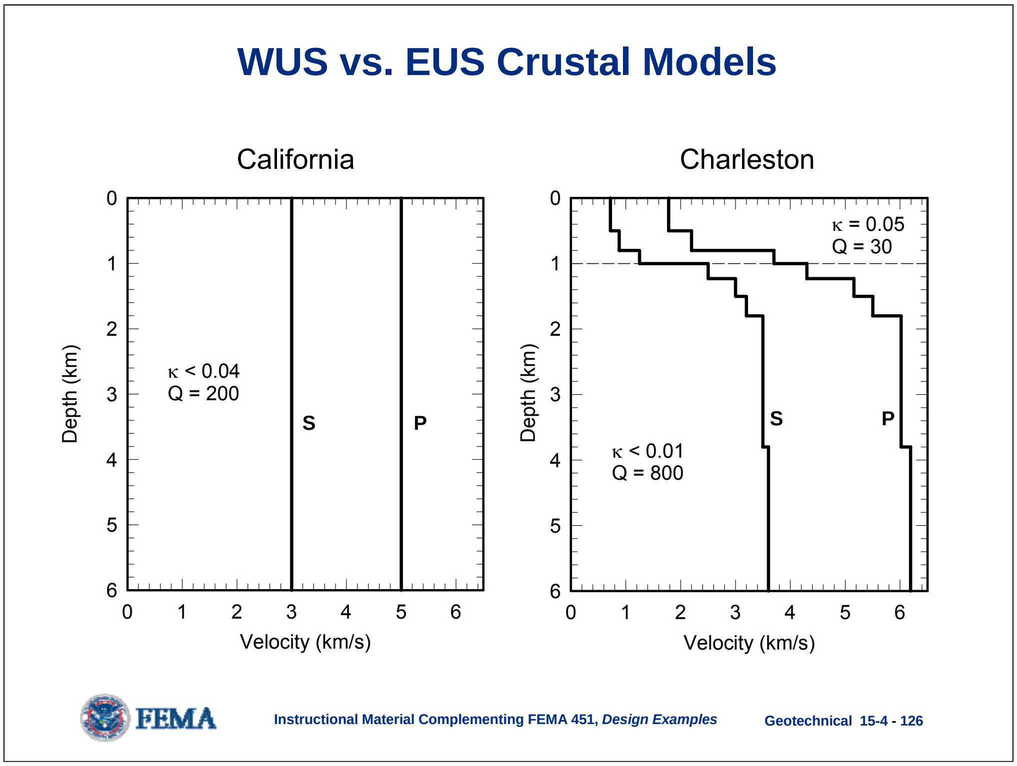
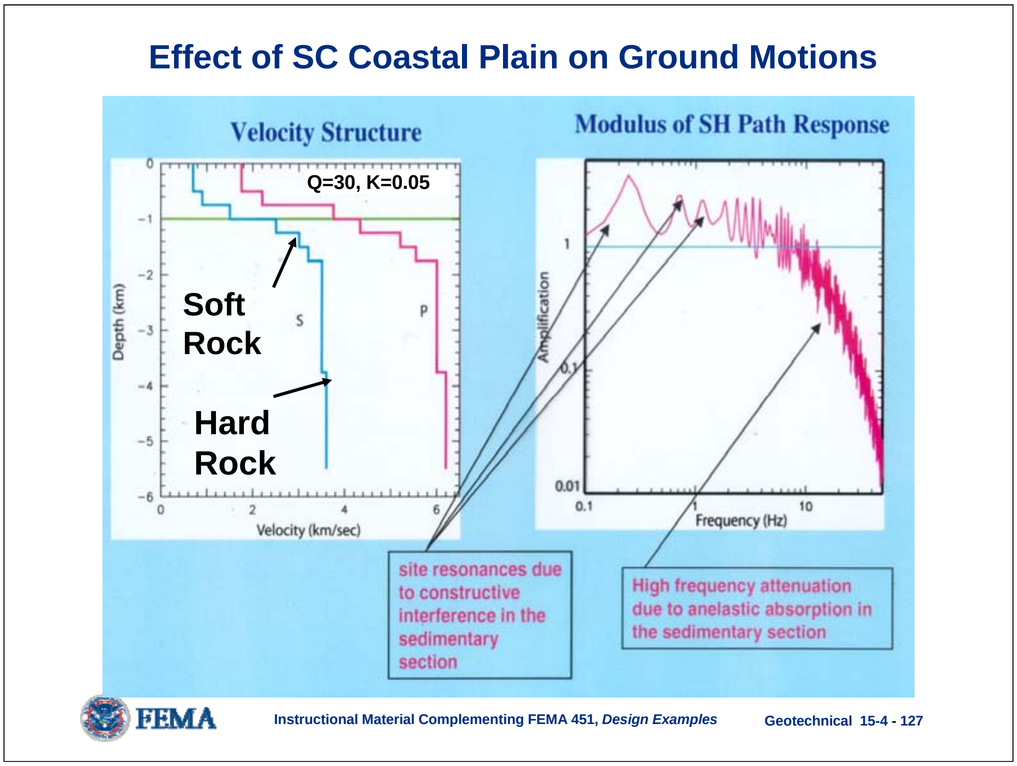
None.

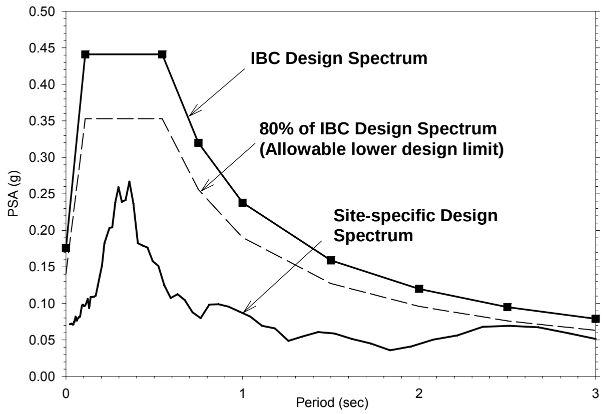


The rock in California is relatively soft (lower Q) relative to that in the eastern United States. The bedrock in the western states is similar to the characteristic of soft rock (sediments like limestone and marls) in the eastern states.



Slide courtesy of Martin Chapman, department of Geological Sciences, Virginia Tech. The de-amplification of the motions by the sediments can be seen (above about 10 Hz).

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

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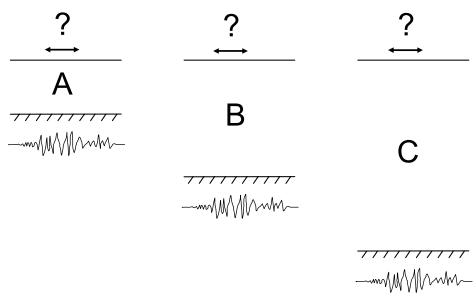
Note the site-specific analysis indicates much less shaking than the IBC predicts due to the USGS maps not accounting for specific geologic conditions in this region.

Special Comments on Site Response Analysis in CEUS

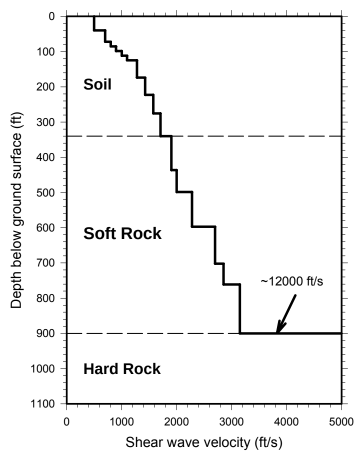
- Analysis techniques common in WUS, may not apply in many cases in CEUS
- Site response (i.e., SHAKE) analyses not as straight-forward in CEUS
- SHAKE has depth limitations (600 ft.? CEUS sites can be deeper)
- Where is halfspace? (Vs = 2000 ft/sec rule of thumb not always applicable in CEUS)

Modeling a deeper profile is better if base motion is appropriately modified and if damping is not too high (may want to use flat degradation curves in SHAKE for deep soil layers).

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.

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Again, a deeper profile is probably better to use if the base motions are appropriately modified and if damping is not too high (may want to use flat degradation curves in SHAKE for deep soil layers). Problem is in cases such as in sediment basins in the central United States where the rock velocities continue to increase with depth for 1 km or more. The assumption of a halfspace implies no velocity increase below and as such the half pace must be either placed at great depth or in a location where the effect of this violation of the basic halfspace assumption is minimized.

A Final Point to Remember….

Relative PGAs in the United States

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The hazard levels in the CUES cities begin to approach some of those in the WUS when the soil conditions are considered. Slide is for illustrative purposed only. The 500 year event motions are used in this case.

Soil is the great equalizer:

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Note that the hazard levels in the CUES cities begin to approach some of those in the WUS when the soil conditions are considered. Slide is for illustrative purposed only. The 500 year event motions are used in this case.

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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None.

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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None.