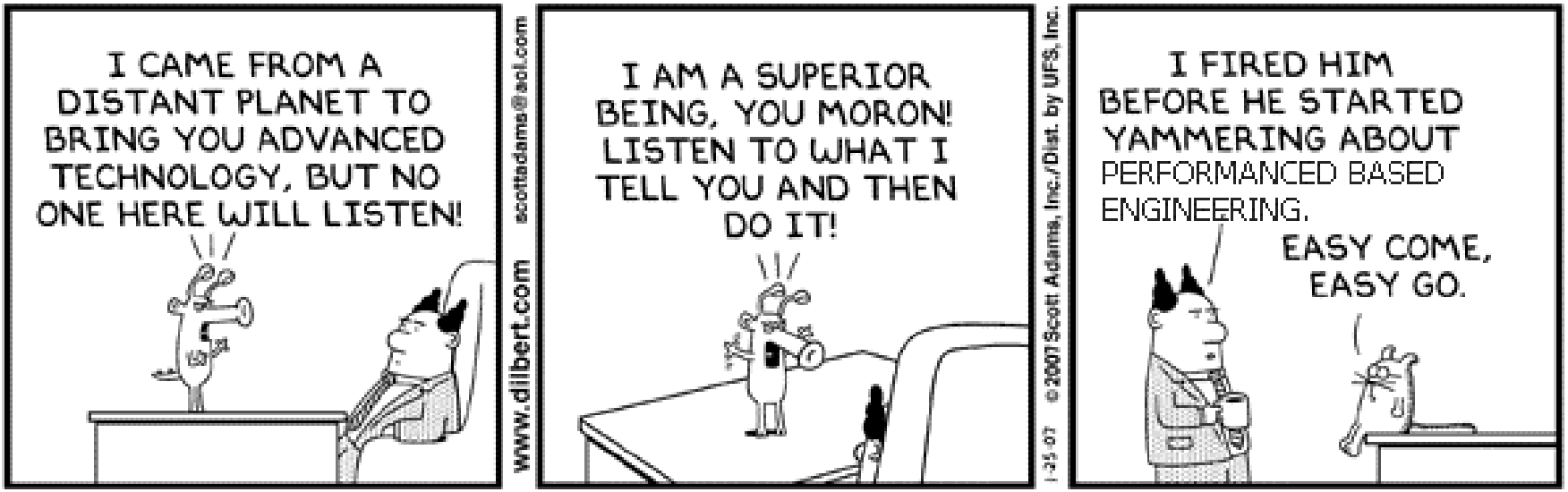
Impact of Foundation Response on the Retrofit of an Existing Hospital

Marshall Lew

MACTEC Engineering & Consulting

Changing the profession?



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With apologies to Scott Adams and Dilbert.

Traditional Geotechnical Practice

- Wide-spread use of empirical formulae and values
- Correlation of Index values to poorly controlled field tests
- Use of poorly understood and not consistently applied factors of safety
- Uncertainty in degrees of conservatism

Traditional Geotechnical Practice

- Based on Allowable Stresses, not LRFD
- Governed by Settlement
- Compounding of factors of safety

  F.S. = $(1.1)^4$ = 1.5

  - H. Bolton Seed, circa 1972
- Lack of precision in making guesstimates

Performance Based Geotechnical Engineering

- Need for reduction in uncertainties
- Forget the concept of factor of safety
- Reliance on better testing methods and less reliance on empirical correlations
- Need to define dispersion or uncertainty
- Close interaction and iterative communication between Structural and Geotechnical Engineers
- Need to guesstimate with precision

Performance Based Geotechnical Engineering

- Why do we need to do this?
  - To predict the actual behavior (or range of possible behaviors) of the structure
  - To understand the potential failure mechanism(s)
  - To have consistency with design of superstructure
  - To have increased confidence in design
  - To have potential cost savings

Shallow Foundation Analysis

- Define behavior by use of foundation stiffnesses
- Example of bi-linear (elasto-plastic) foundation stiffnesses for a retrofit of an existing hospital building

Modeling of Foundation Stiffness (FEMA 356/ASCE 41)

Considering

Inertial Effects Only

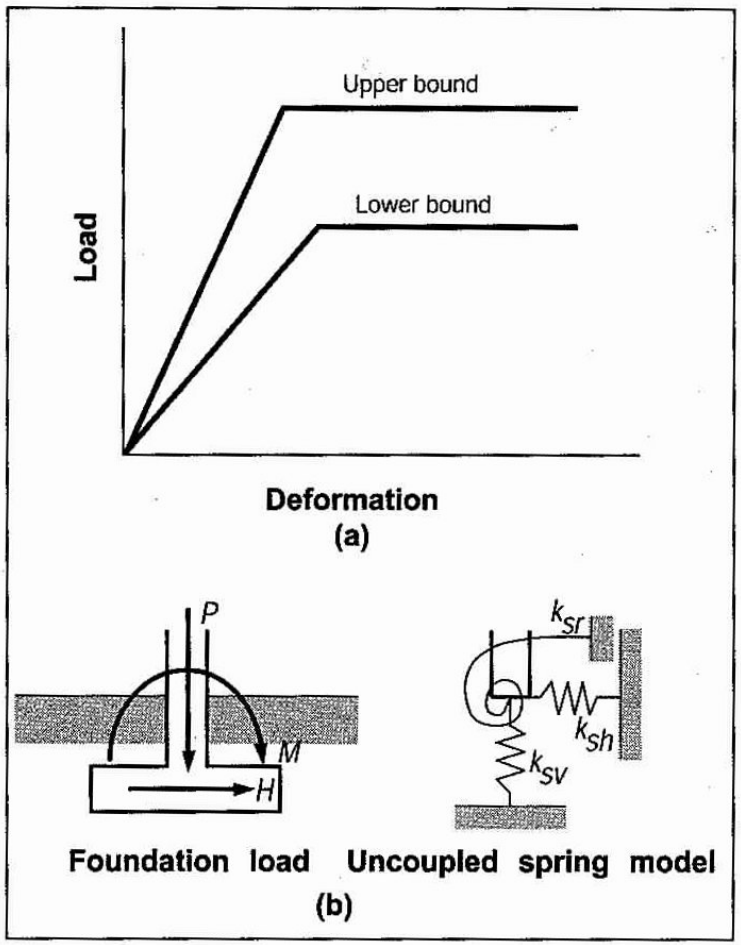
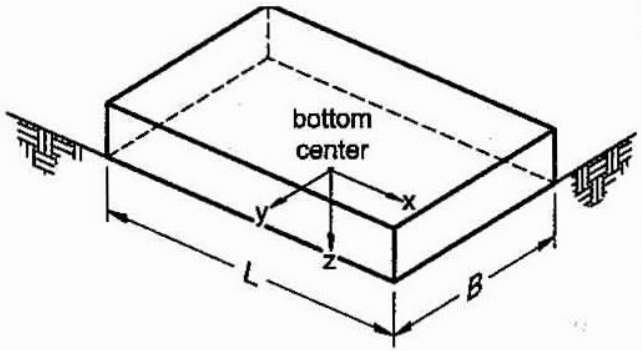


Figure 4-3 *(a) Idealized Elasto-Plastic Load-Deformation Behavior for Soils (b) Uncoupled Spring Model for Rigid Footings*

Modeling of Foundation Stiffness (FEMA 356/ASCE 41)

Chapter 4: Foundations and Geologic Site Hazards

Degree of Freedom	Stiffness of Foundation at Surface	Note
Translation along x-axis	$K_{x,sur} = \frac{GB}{2-\nu}\left[3.4\left(\frac{L}{B}\right)^{0.65} + 1.2\right]$	
Translation along y-axis	$K_{y,sur} = \frac{GB}{2-\nu}\left[3.4\left(\frac{L}{B}\right)^{0.65} + 0.4\frac{L}{B} + 0.8\right]$	
Translation along z-axis	$K_{z,sur} = \frac{GB}{1-\nu}\left[1.55\left(\frac{L}{B}\right)^{0.75} + 0.8\right]$	
Rocking about x-axis	$K_{xx,sur} = \frac{GB^3}{1-\nu}\left[0.4\left(\frac{L}{B}\right) + 0.1\right]$	
Rocking about y-axis	$K_{yy,sur} = \frac{GB^3}{1-\nu}\left[0.47\left(\frac{L}{B}\right)^{2.4} + 0.034\right]$	Orient axes such that $L \geq B$
Torsion about z-axis	$K_{zz,sur} = GB^3\left[0.53\left(\frac{L}{B}\right)^{2.45} + 0.51\right]$	

Modeling of Foundation Stiffness (FEMA 356/ASCE 41)

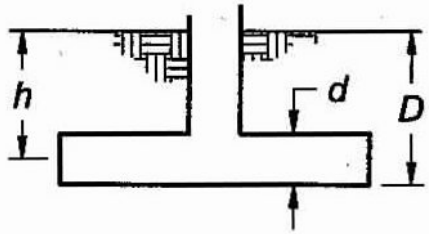
Degree of Freedom	Correction Factor for Embedment	Note
Translation along x-axis	$\beta_x = \left(1 + 0.21\sqrt{\frac{D}{B}}\right) \cdot \left[1 + 1.6\left(\frac{hd(B+L)}{BL^2}\right)^{0.4}\right]$	 d = height of effective sidewall contact (may be less than total foundation height) h = depth to centroid of effective sidewall contact
Translation along y-axis	$\beta_y = \beta_x$	
Translation along z-axis	$\beta_z = \left[1 + \frac{1}{21}\frac{D}{B}\left(2 + 2.6\frac{B}{L}\right)\right] \cdot \left[1 + 0.32\left(\frac{d(B+L)}{BL}\right)^{2/3}\right]$	
Rocking about x-axis	$\beta_{xx} = 1 + 2.5\frac{d}{B}\left[1 + \frac{2d}{B}\left(\frac{d}{D}\right)^{-0.2}\sqrt{\frac{B}{L}}\right]$	
Rocking about y-axis	$\beta_{yy} = 1 + 1.4\left(\frac{d}{L}\right)^{0.6}\left[1.5 + 3.7\left(\frac{d}{L}\right)^{1.9}\left(\frac{d}{D}\right)^{-0.6}\right]$	For each degree of freedom, calculate $K_{emb} = \beta K_{sur}$
Torsion about z-axis	$\beta_{zz} = 1 + 2.6\left(1 + \frac{B}{L}\right)\left(\frac{d}{B}\right)^{0.9}$	

Figure 4-4 ***Elastic Solutions for Rigid Footing Spring Constraints***

Modeling of Foundation Stiffness (FEMA 356/ASCE 41)

Considering
- Inertial Effects Only

We need to estimate:

1. Initial Stiffness
2. Ultimate Load Capacity

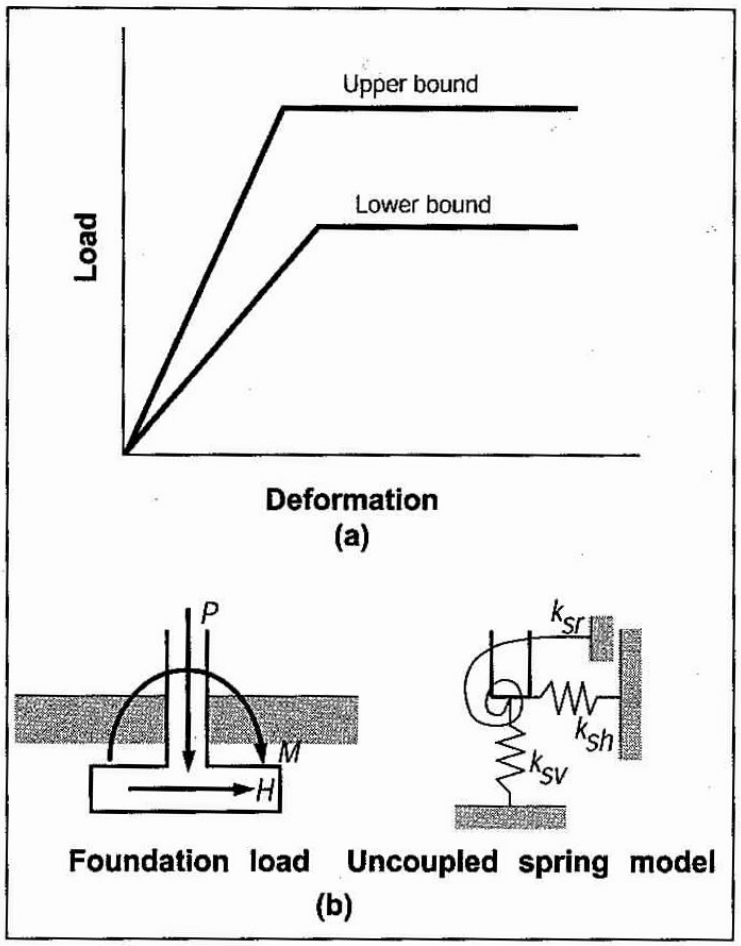


Figure 4-3 *(a) Idealized Elasto-Plastic Load-Deformation Behavior for Soils (b) Uncoupled Spring Model for Rigid Footings*

1. Initial Foundation Stiffness

4.4.2.1 Shallow Bearing Foundations

4.4.2.1.1 Stiffness Parameters

The initial shear modulus, G_o, shall be calculated in accordance with Equation (4-4) or (4-5) where v_s is the shear wave velocity at low strains, γ is the weight density of the soil, and g is the acceleration due to gravity.

$$G_o = \frac{\gamma v_s^2}{g} \qquad (4\text{-}4)$$

1. Initial Foundation Stiffness

$$G_o \cong 20{,}000\,(N_1)_{60}^{1/3}\sqrt{\sigma'_o}$$

where:

$(N_1)_{60}$	=	Standard Penetration Test blow count normalized for an effective stress of 1.0 ton per square foot confining pressure and corrected to an equivalent hammer energy efficiency of 60%
σ'_o	=	Effective vertical stress in psf
σ'_o	=	$\gamma_t d - \gamma_w (d - d_w)$
γ_t	=	Total unit weight of soil
γ_w	=	Unit weight of water
d	=	Depth to sample
d_w	=	Depth to ground-water level

G_o in Equation (4-5) is expressed in pounds per square foot, as is σ'_o.

1. Initial Foundation Stiffness

Table 4-7 Effective Shear Modulus Ratio (G/G_0)

Site Class	Effective Peak Acceleration, $S_{XS}/2.5$			
	$S_{XS}/2.5$ =0	$S_{XS}/2.5$ =0.1	$S_{XS}/2.5$ =0.4	$S_{XS}/2.5$ =0.8
A	1.00	1.00	1.00	1.00
B	1.00	1.00	0.95	0.90
C	1.00	0.95	0.75	0.60
D	1.00	0.90	0.50	0.10
E	1.00	0.60	0.05	*
F	*	*	*	*

Note: Use straight-line interpolation for intermediate values of $S_{XS}/2.5$.

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

2. Ultimate Bearing Capacities

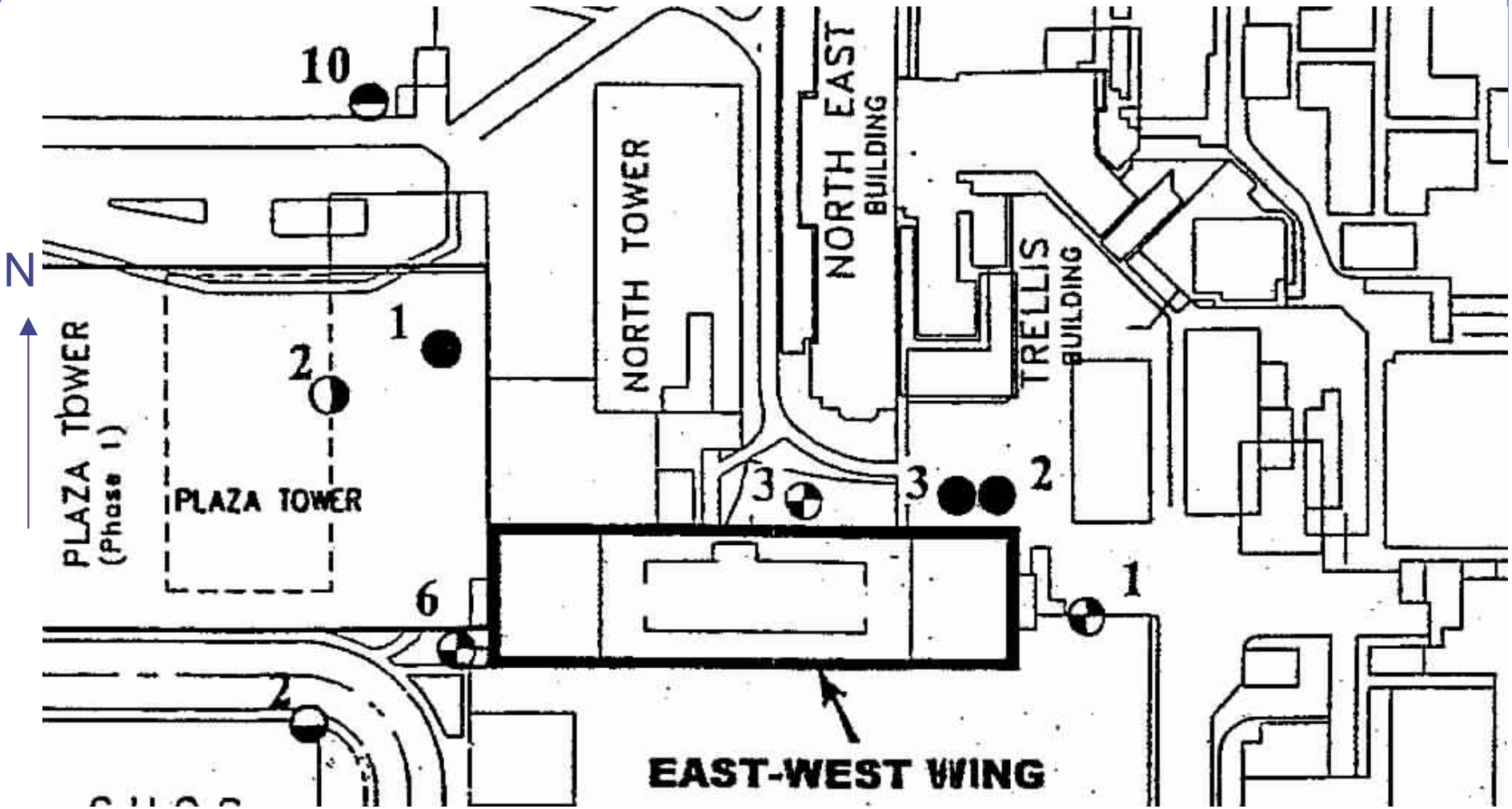
Presumptive Foundation Capacities

Table 4-2 ***Parameters for Calculating Presumptive Expected Foundation Load Capacities of Spread Footings and Mats***

Class of Materials[2]	Vertical Foundation Pressure[3] Lbs/Sq. Ft. (q_c)	Lateral Bearing Pressure Lbs/Sq. Ft./Ft of Depth Below Natural Grade[4]	Lateral Sliding[1]	
			Coefficient[5]	Resistance[6] Lbs/Sq. Ft
Massive Crystalline Bedrock	8000	2400	0.80	—
Sedimentary and Foliated Rock	4000	800	0.70	—
Sandy Gravel and/or Gravel (GW and GP)	4000	400	0.70	—
Sand, Silty Sand, Clayey Sand, Silty Gravel, and Clayey Gravel (SW, SP, SM, SC, GM, and GC)	3000	300	0.50	—
Clay, Sandy Clay, Silty Clay, and Clayey Silt (CL, ML, MH, and CH)	2000[7]	200	—	260

1. Lateral bearing and lateral sliding resistance shall be permitted to be combined.
2. For soil classifications OL, OH, and PT (i.e., organic clays and peat), a foundation investigation shall be required.
3. All values of expected bearing capacities are for footings having a minimum width of 12 inches and a minimum depth of 12 inches into natural grade. Except where Footnote 7 applies, an increase of 20% is allowed for each additional foot of width or depth to a maximum value of three times the designated value.
4. Shall be permitted to be increased by the amount of the designated value for each additional foot of depth to a maximum of 15 times the designated value.
5. Coefficient applied to the dead load.
6. Lateral sliding resistance value to be multiplied by the contact area. In no case shall the lateral sliding resistance exceed one-half of the dead load.
7. No increase for width shall be permitted.

Example – "East-West" Wing of Existing Hospital



East-West Wing Building Background



Soil and Geologic Conditions

- Natural soils consist of medium dense to very dense silty sand, stiff to hard sandy silt, dense to very dense sand, very stiff silty clay
- Ground water encountered at depth of 43 feet below the ground surface
- Site Class “D”
- Not in a State of California Liquefaction or Landslide Hazard Zone
- Not within an Alquist-Priolo Earthquake Fault Zone

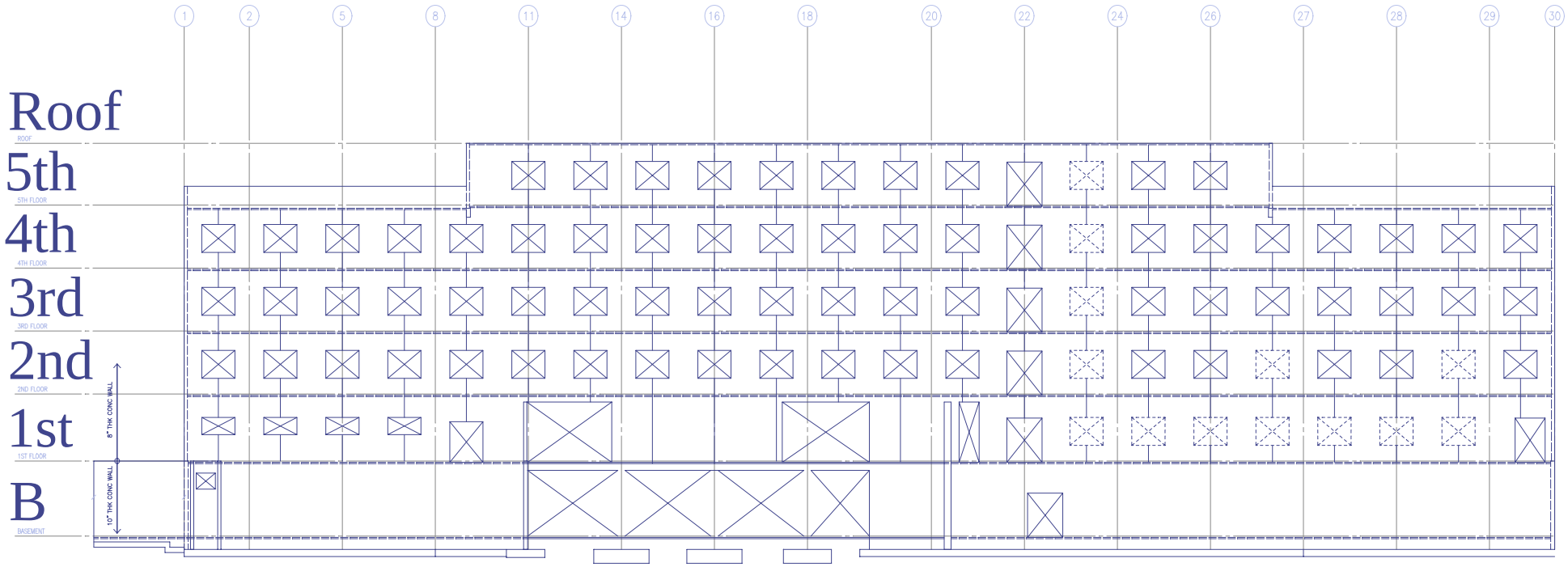
Building Description

- Reinforced concrete
- 4-stories (with partial penthouse) with 1 level basement
- Supported on spread (isolated and wall) footings
- Basement is about 12 feet deep
- Wall footings vary from 2 feet to 7 foot-6 inches wide
- Wall footings are established about 4-1/2 feet below the basement floor.

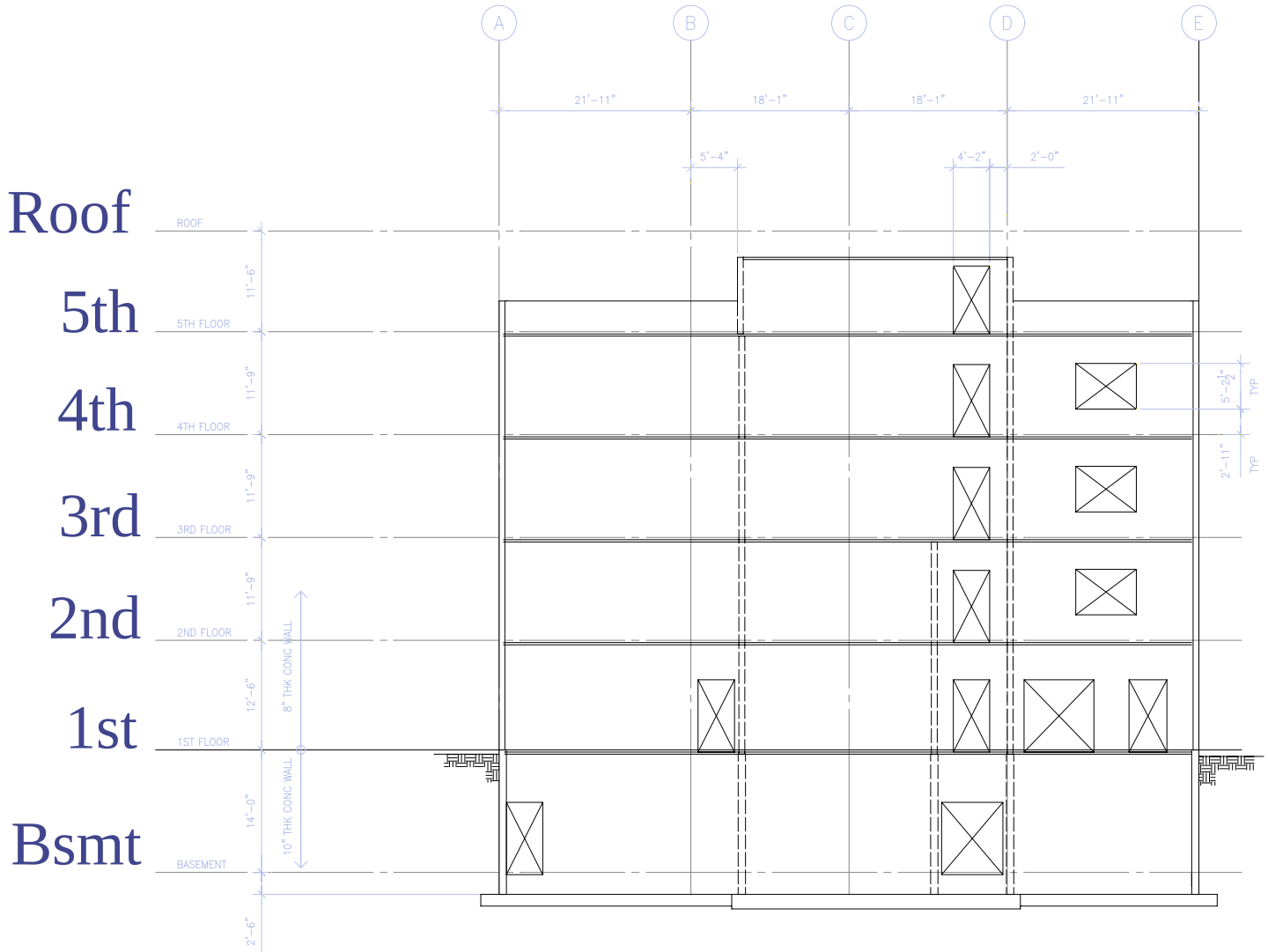
East-West Wing Building Background

- Existing SPC-1 Structural System
- Constructed in 1962
- 5 Stories + Basement
- Lateral System
  - Perforated Reinforced Concrete Wall System (Bearing and Shear) - 8”, 9”, and 10” walls
  - 3” Concrete Floor Slab with Pan Joists
  - Continuous and Isolated Spread Concrete Footings

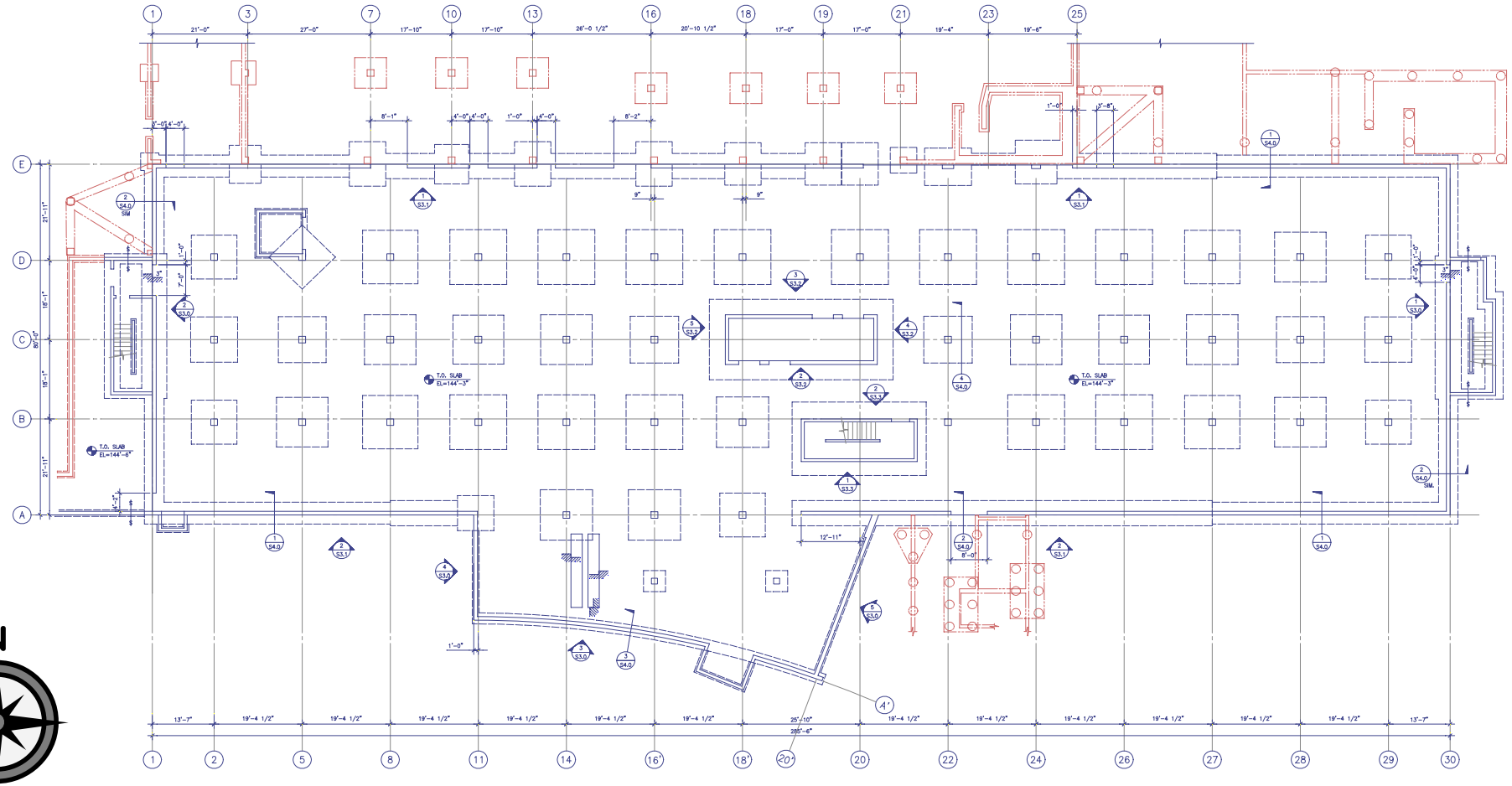
East-West Wing Building Background - North Wall Elevation



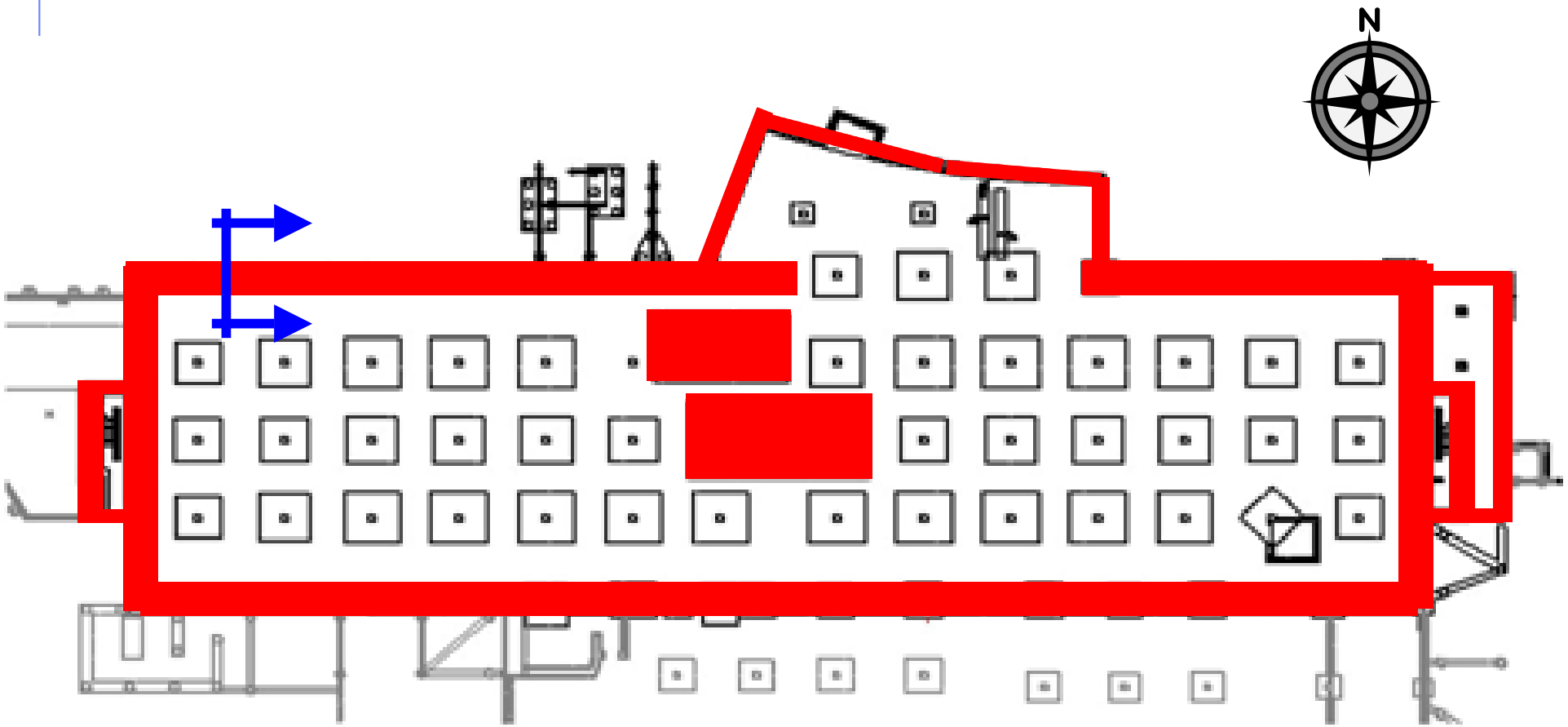
East-West Wing Building Background - East Wall Elevation



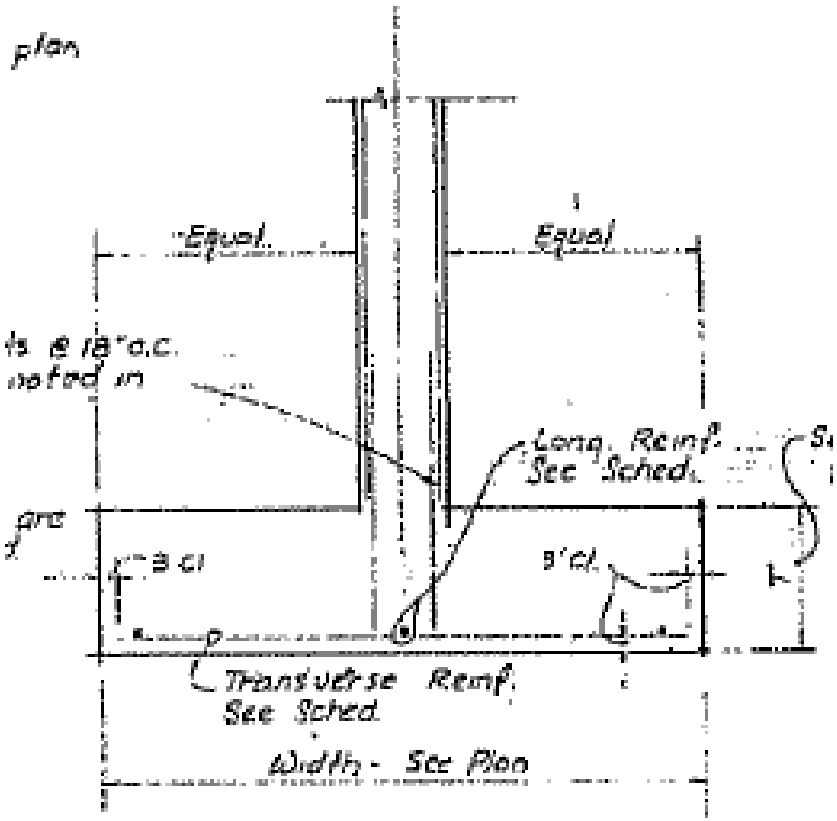
East-West Wing Building Background - Foundation Plan



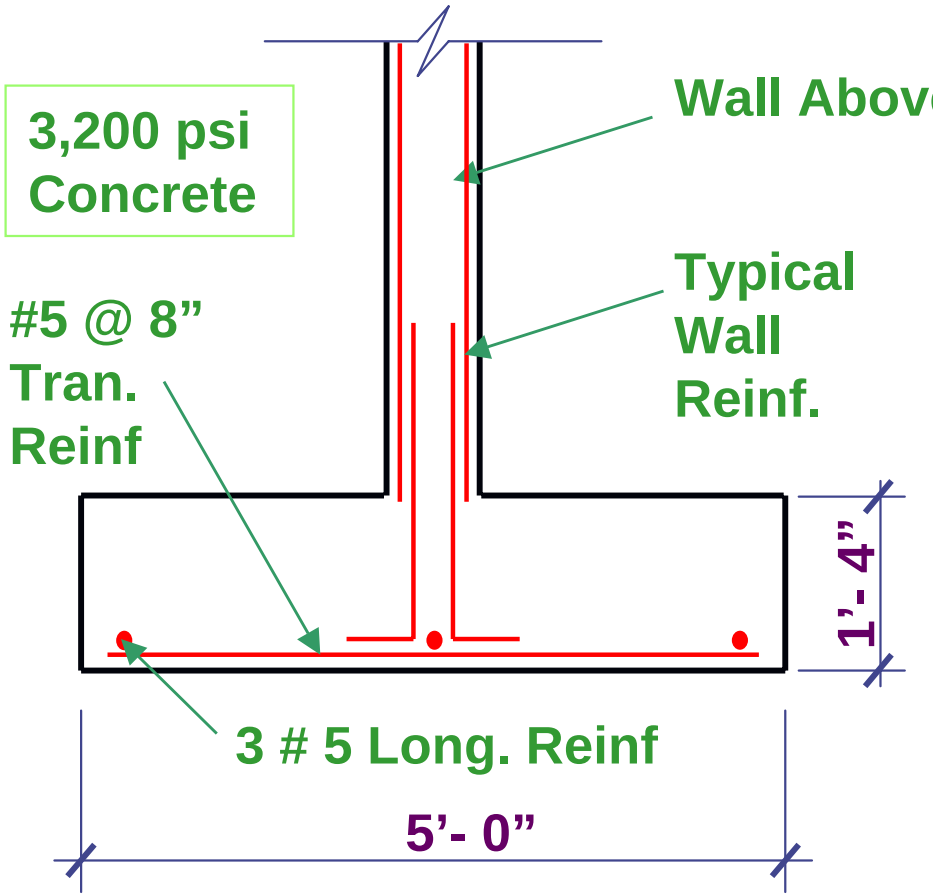
Foundation Plan



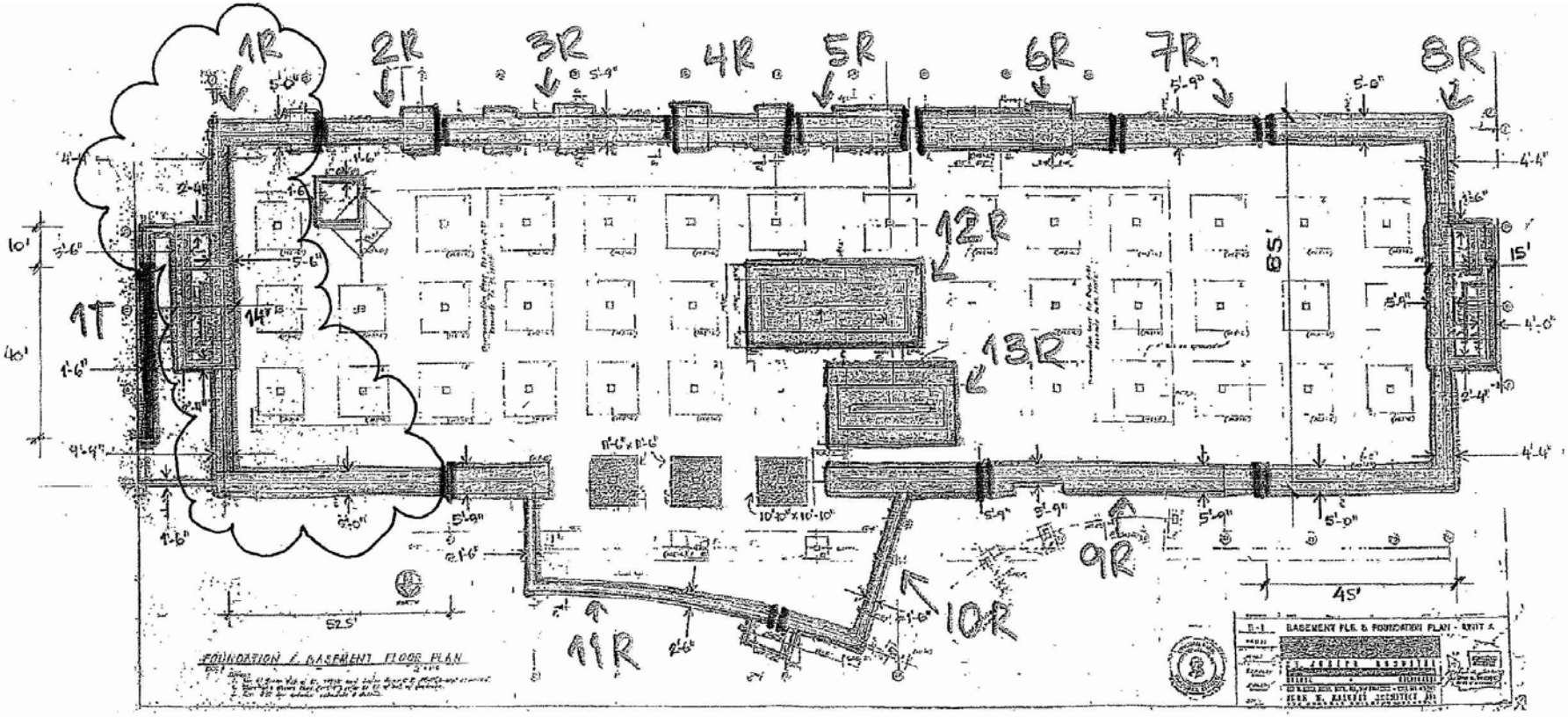
Typical Wall Foundation Detail



TYPICAL REINFORCED WALL FOOTING



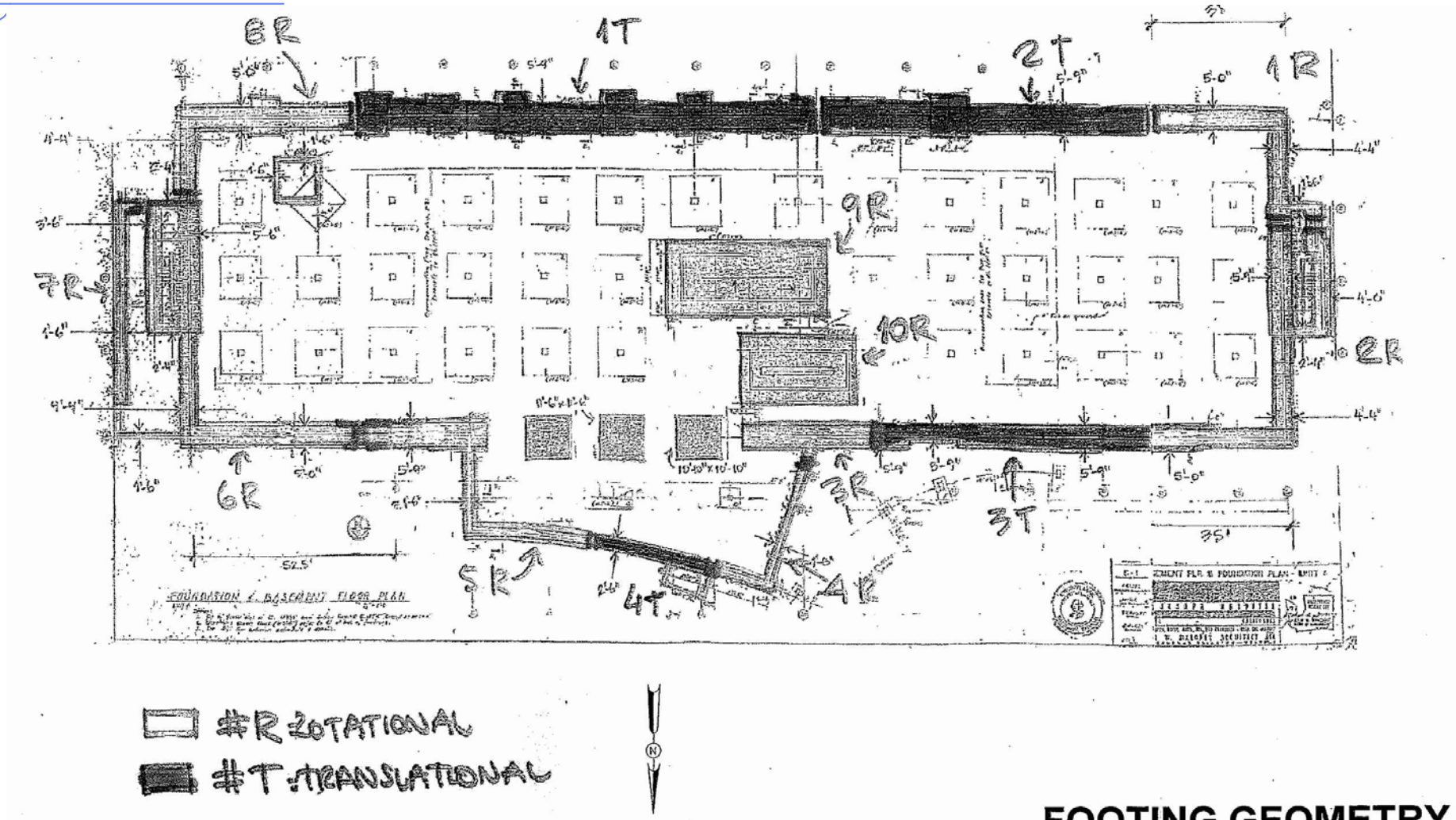
East-West Direction of Shaking



Note change in direction of North arrow.

FOOTING GEOMETRY
East-West Direction of Shaking

North-South Direction of Shaking



FOOTING GEOMETRY
North–South Direction of Shaking

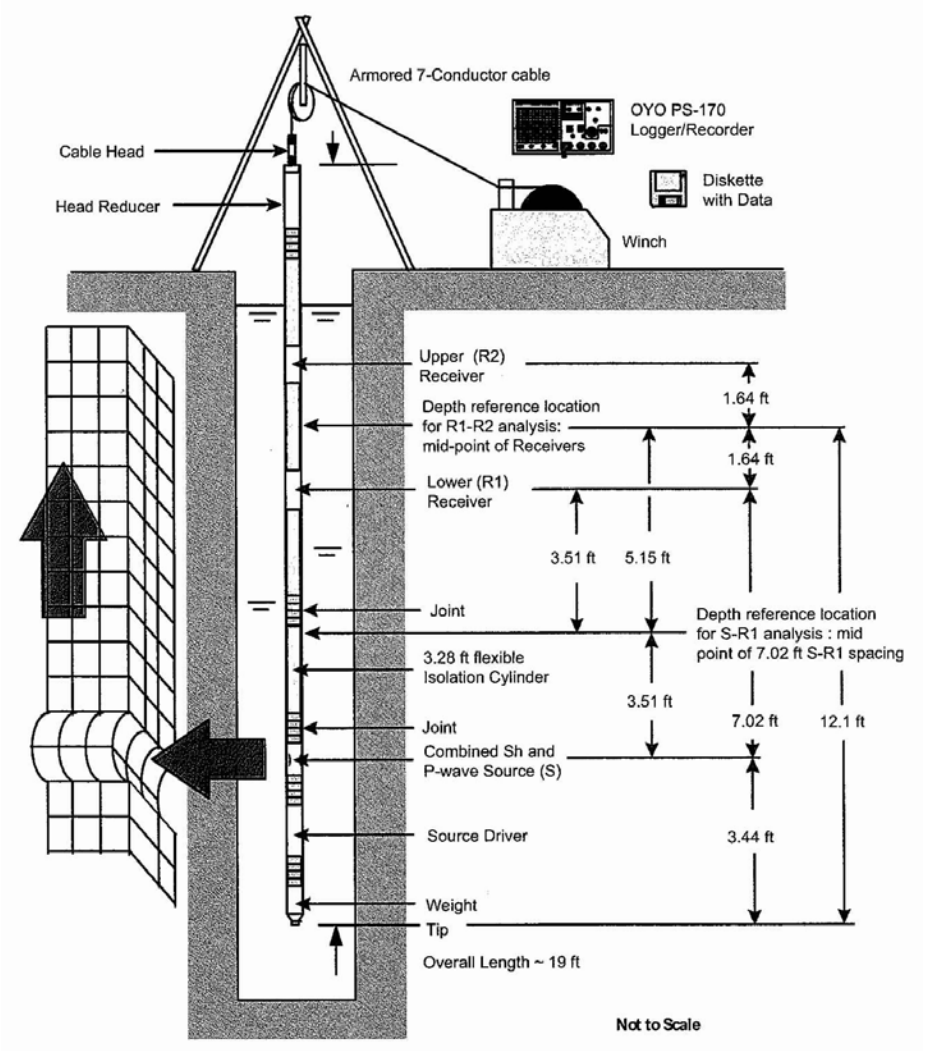
1. Estimate Initial Stiffness

- Estimate the Initial Stiffness by use of published data for general soil types (NOT RECOMMENDED)
- Perform laboratory testing (NOT RECOMMENDED)
- Perform in situ testing

1. Estimate Initial Stiffness

Estimate the Initial Stiffness by use of Suspension Logging of P- and S-waves

$$G_o = \frac{\gamma v_s^2}{g}$$



1. Estimate Initial Stiffness

- Use in situ testing to estimate the Initial Stiffness by use of Suspension Logging of S-waves

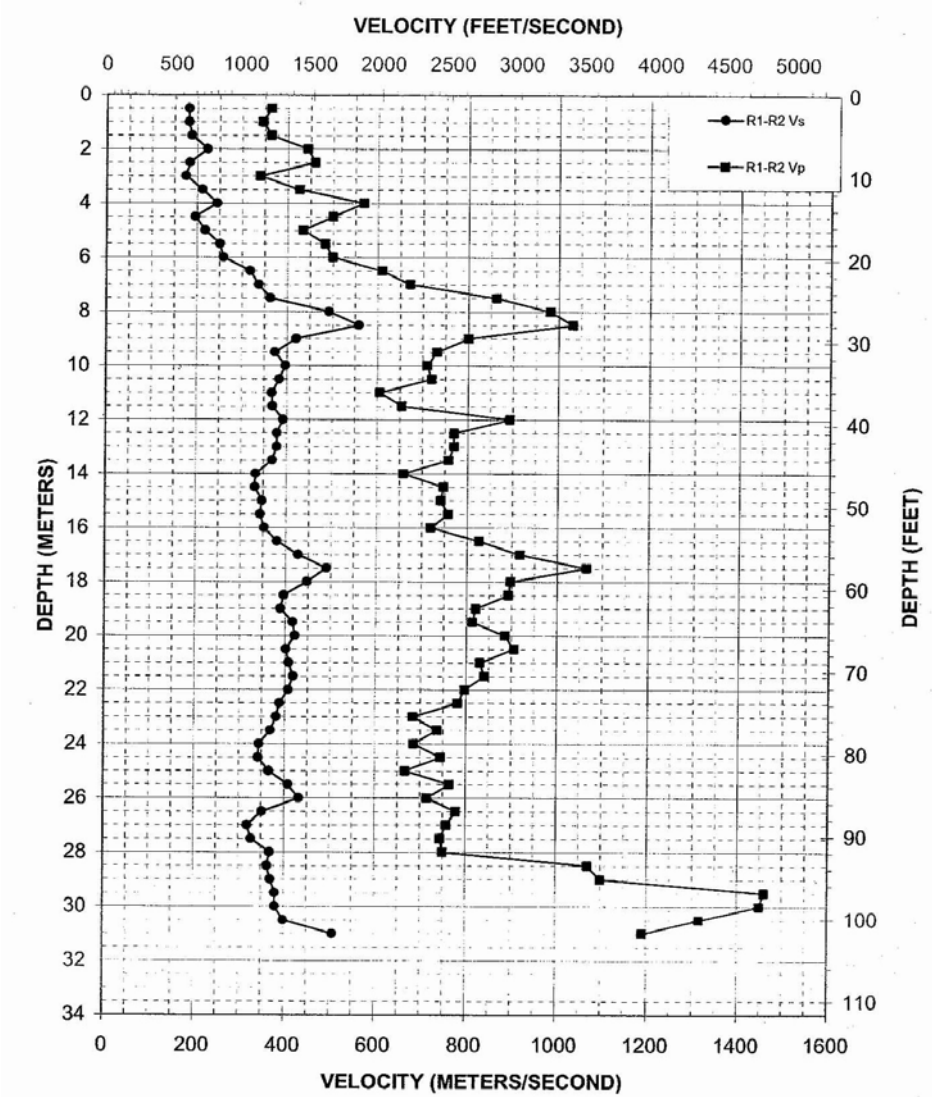


Figure 4. Boring B-2, Suspension P- and S_H-wave velocities

1. Estimate Initial Stiffness

- Estimate the shear modulus, G_o, at low strains
- $G_o = \rho v_s^2$

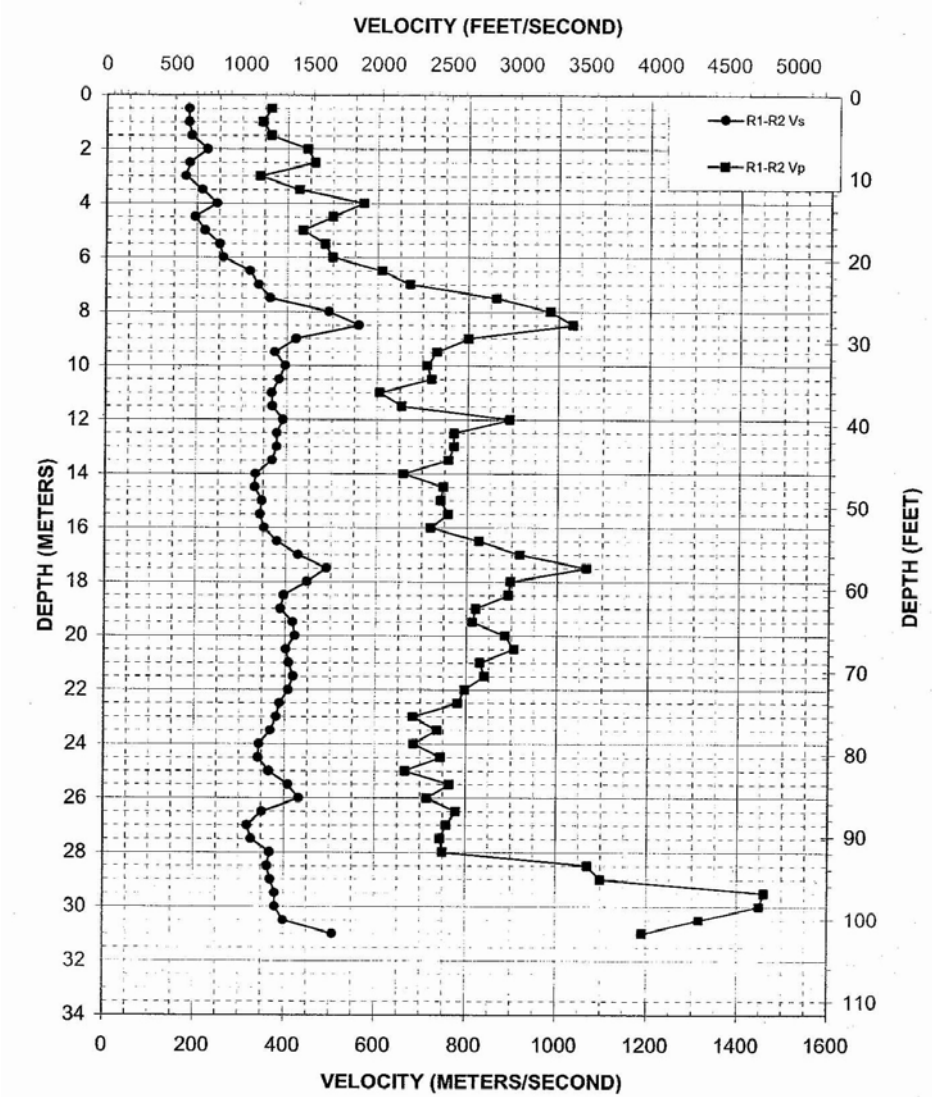


Figure 4. Boring B-2, Suspension P- and S_H-wave velocities

1. Estimate Initial Stiffness

- Use v_s directly under the bottom of footing
- Consider v_s to "depth" of ½ to 1 footing width below bottom of footing
- Do not use v_{s30}!!!

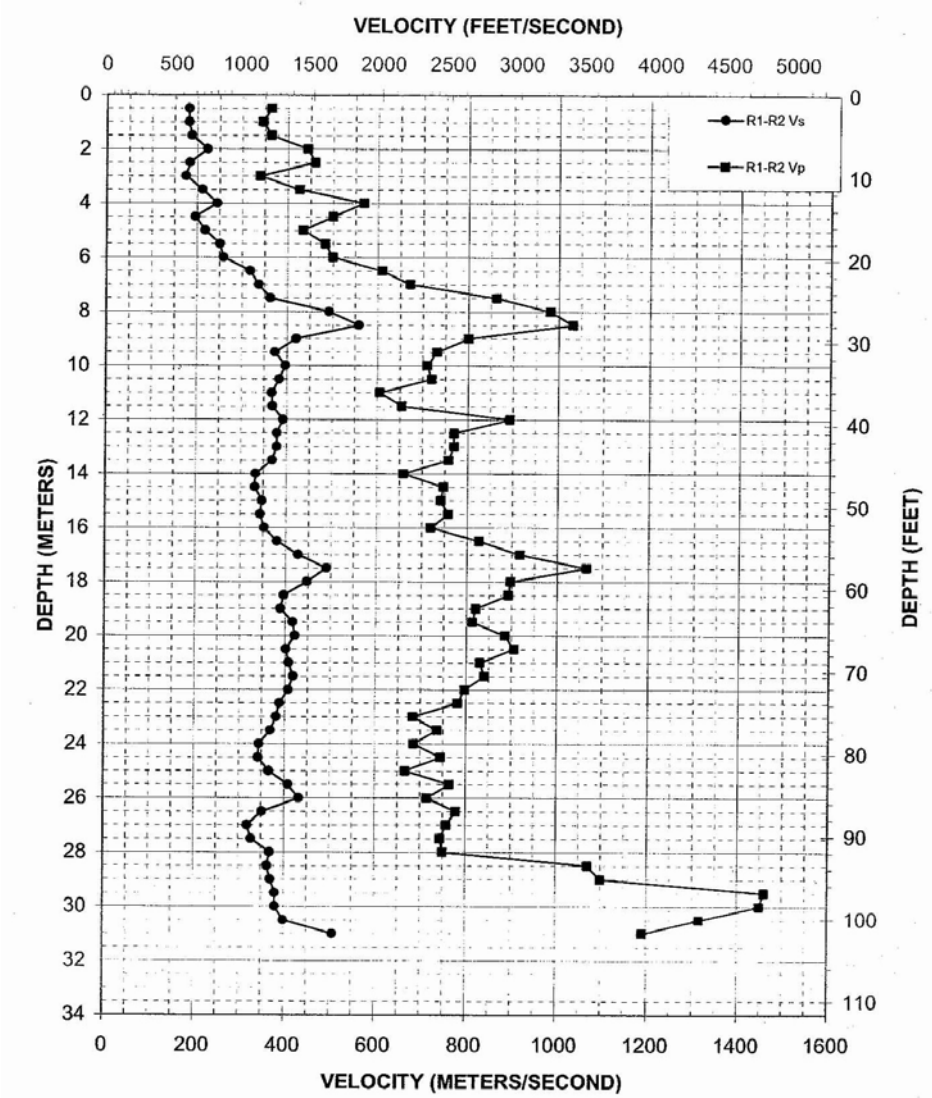


Figure 4. Boring B-2, Suspension P- and S_H-wave velocities

1. Estimate Initial Stiffness – Variability of the initial stiffness

One method to account for spatial variability of the shear wave velocity is based on studies reported in G.R. Toro (1997), "Probabilistic Models of Shear Wave Velocity Profiles at the Savannah River Site, South Carolina," Report to Pacific Engineering and Analysis.

Toro's methodology is reported in:

Stewart et al., Documentation and Analysis of Field Case Histories of Seismic Compression during the 1994 Northridge, California, Earthquake. PEER Report 2002/09, October 2002. (Chapter 7)

http://cee.ea.ucla.edu/faculty/jstewart/Northridge%20Earthquake%20Paper.htm

1. Estimate Initial Stiffness – Variability of the initial stiffness

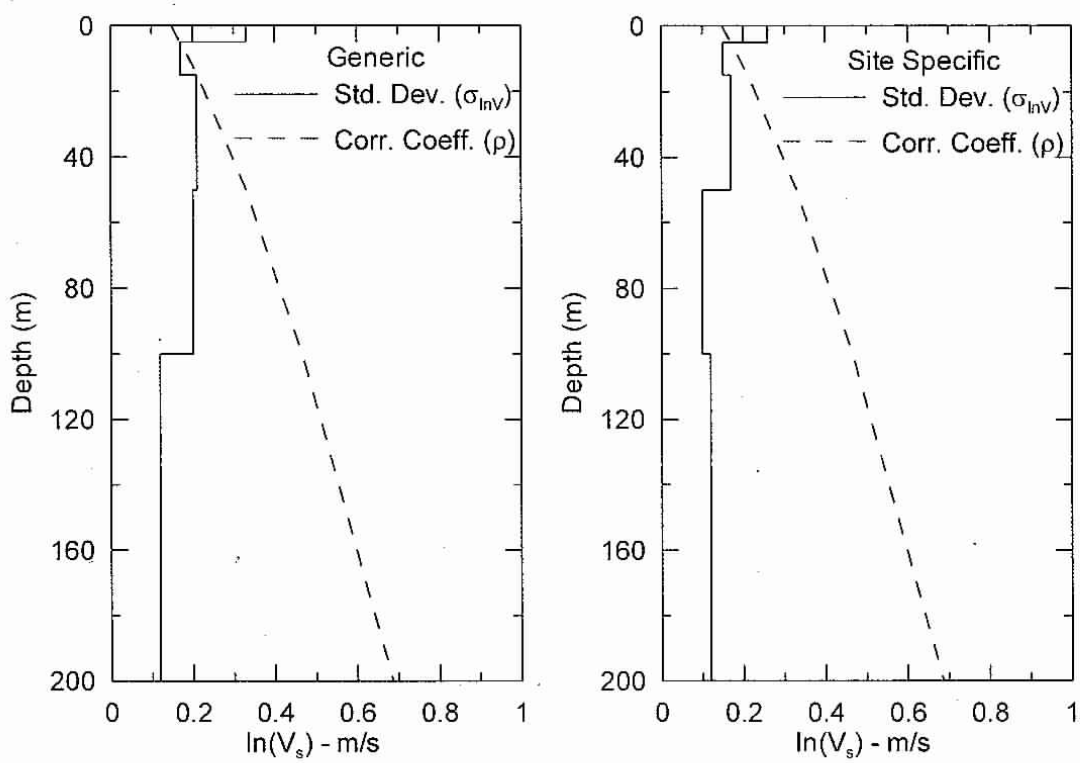


Fig. 7.17. Variation of standard deviation and correlation coefficient with depth for generic and site-specific site profiles (Toro et al. 1997)

Toro (1997) found that shear wave velocities, for a given depth range, were lognormally distributed with a standard deviation that is a function of depth.

1. Estimate Initial Stiffness – Variability of the initial stiffness

- Toro (1997) found that shear wave velocities, for a given depth range, were lognormally distributed with a standard deviation that is a function of depth.
- Expected range of $v_s = v_{s\text{-mean}} (e^{-\sigma}$ to $e^{+\sigma})$
- Where $\sigma = 0.15$ to 0.20 for shallow depths in natural log units

2. Ultimate Bearing Capacities

- Estimate the Ultimate Bearing Capacities of the individual footings using the actual foundation dimensions and depth of embedment.
- The continuous foundations were segmented based on expected behavior during a DBE event. (This is iterative effort and requires interaction between the Geotechnical and Structural Engineers)

2. Ultimate Bearing Capacities

- Ultimate Bearing Capacity for continuous footings by the Terzaghi Equation:

$$q_{ult} = cN_c + \gamma DN_q + 0.5\,\gamma BN_\gamma$$

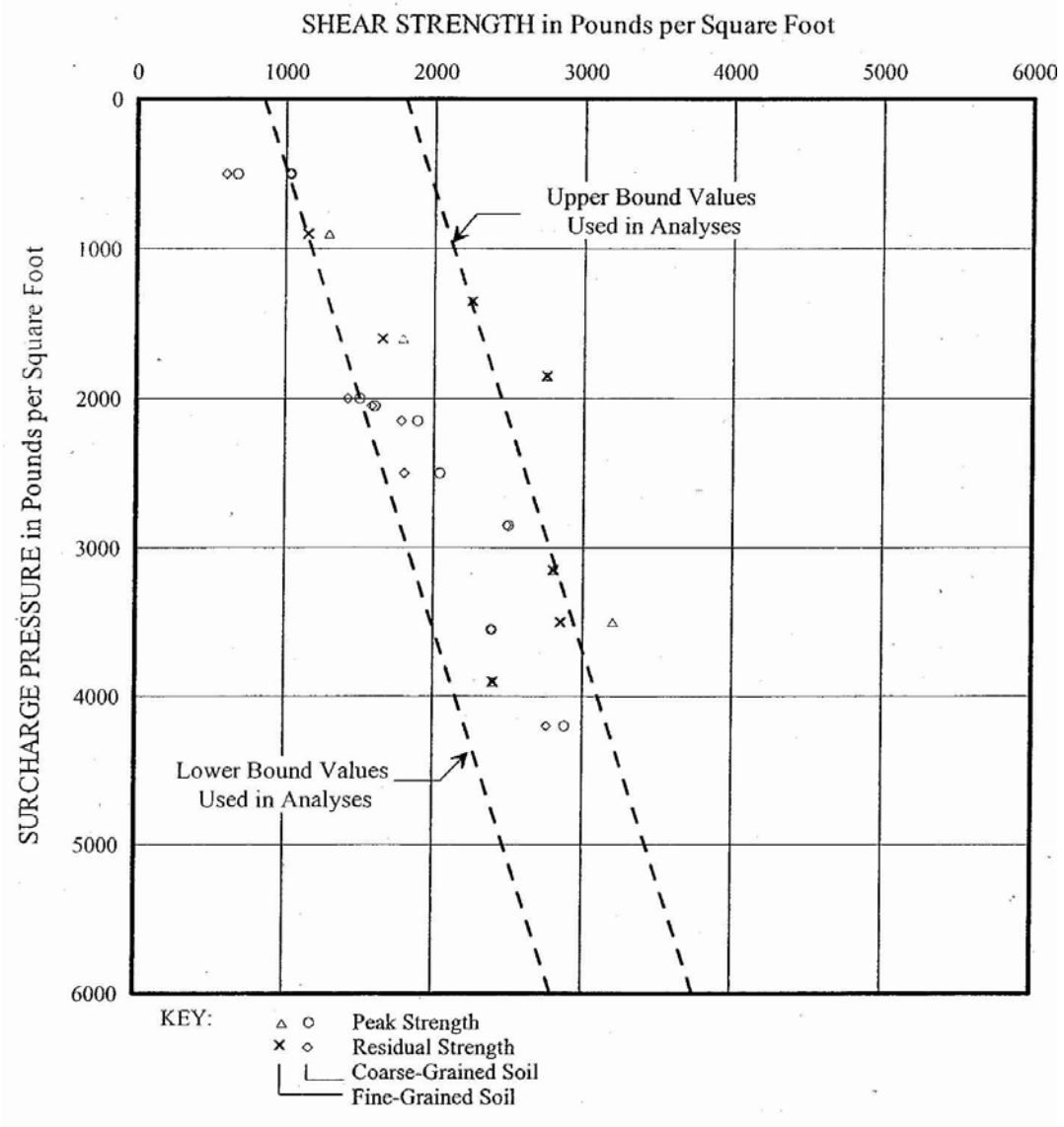
where c = cohesion of soil, γ = wet unit weight of soil, and N_c, N_q and N_γ are Terzaghi bearing capacity factors; and D is the depth of footing embedment and B is the width of the continuous footing.

2. Ultimate Bearing Capacities

- Determine upper and lower bounds of shear shear strength for analysis of ultimate bearing capacity
- Consider the state of the soil to determine the appropriate testing methods for determining shear strength
- For example, soils were unsaturated and not near saturation, so it was considered appropriate to use drained direct shear tests

2. Ultimate Bearing Capacities

- Determine upper and lower bounds of shear strength for analysis of ultimate bearing capacity



2. Ultimate Bearing Capacities

- For Continuous Footings:

  Upper Bound Bearing Capacity in units of psf:

  $$27{,}900 + 743\,D + 242\,B$$

  Lower Bound Bearing Capacity in units of psf:

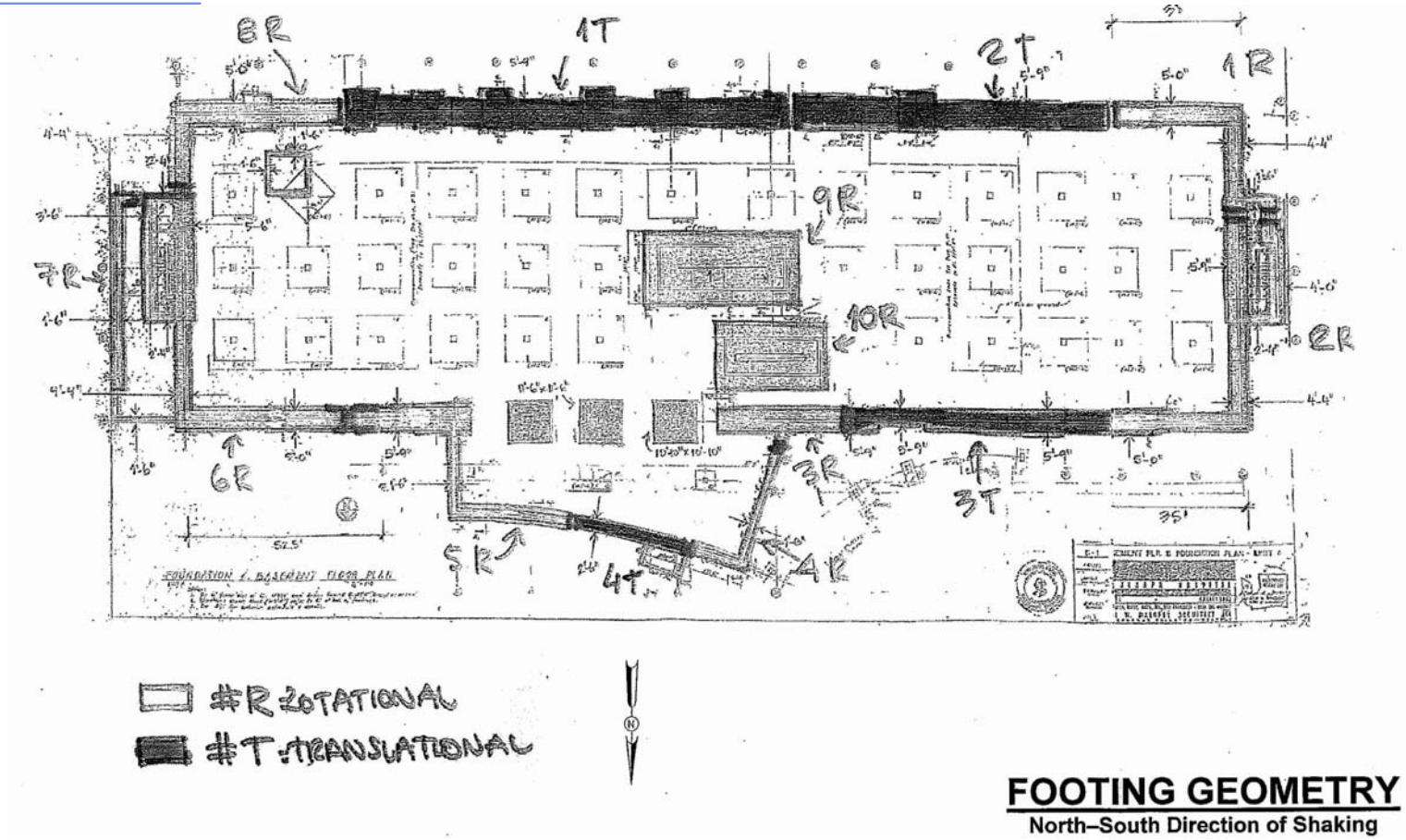
  $$13{,}175 + 743\,D + 242\,B$$

  Where D and B are in units of feet

2. Ultimate Bearing Capacities

- Note: the allowable bearing capacity for original design of building was *4 ksf*.
- If the Factor of Safety is 3, then the ultimate bearing capacity was *12 ksf*.
- If presumptive FEMA 356 allowable bearing capacity of 3 ksf is assumed, then presumptive ultimate bearing capacity would be about 9 ksf (assuming FS = 3).

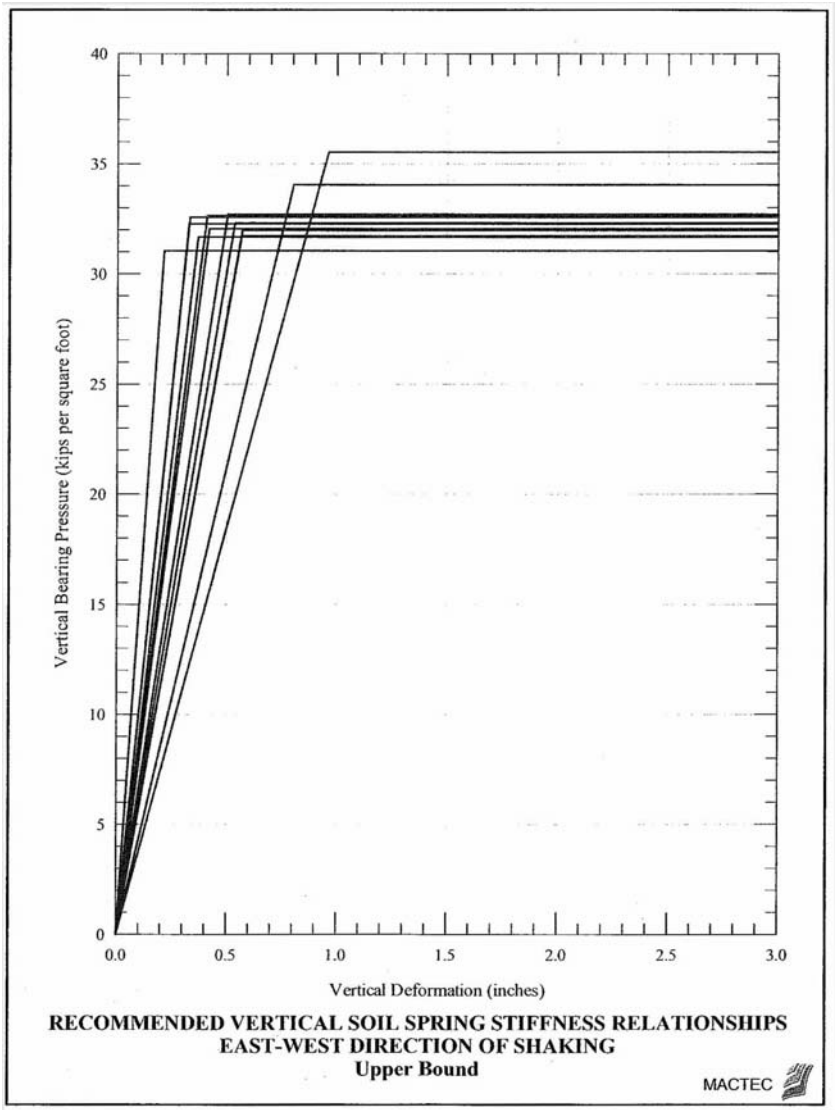
Example – North-South Direction of Shaking

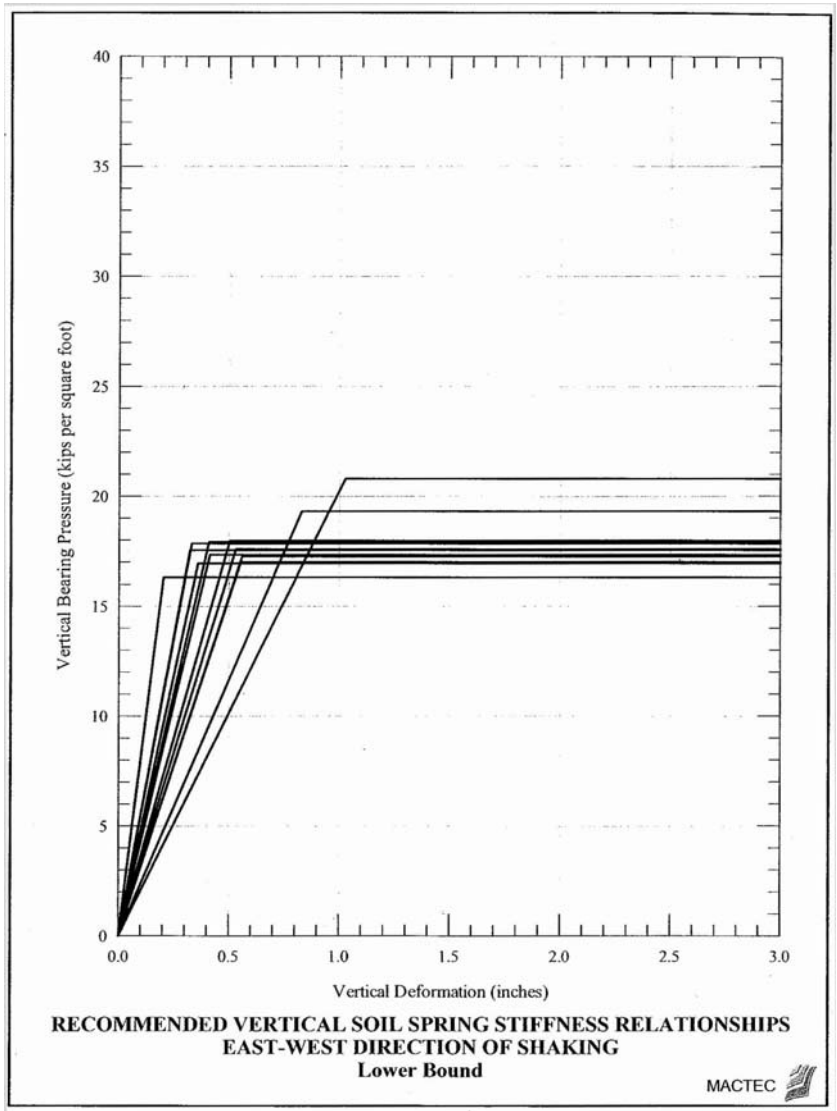


FOOTING GEOMETRY
North–South Direction of Shaking

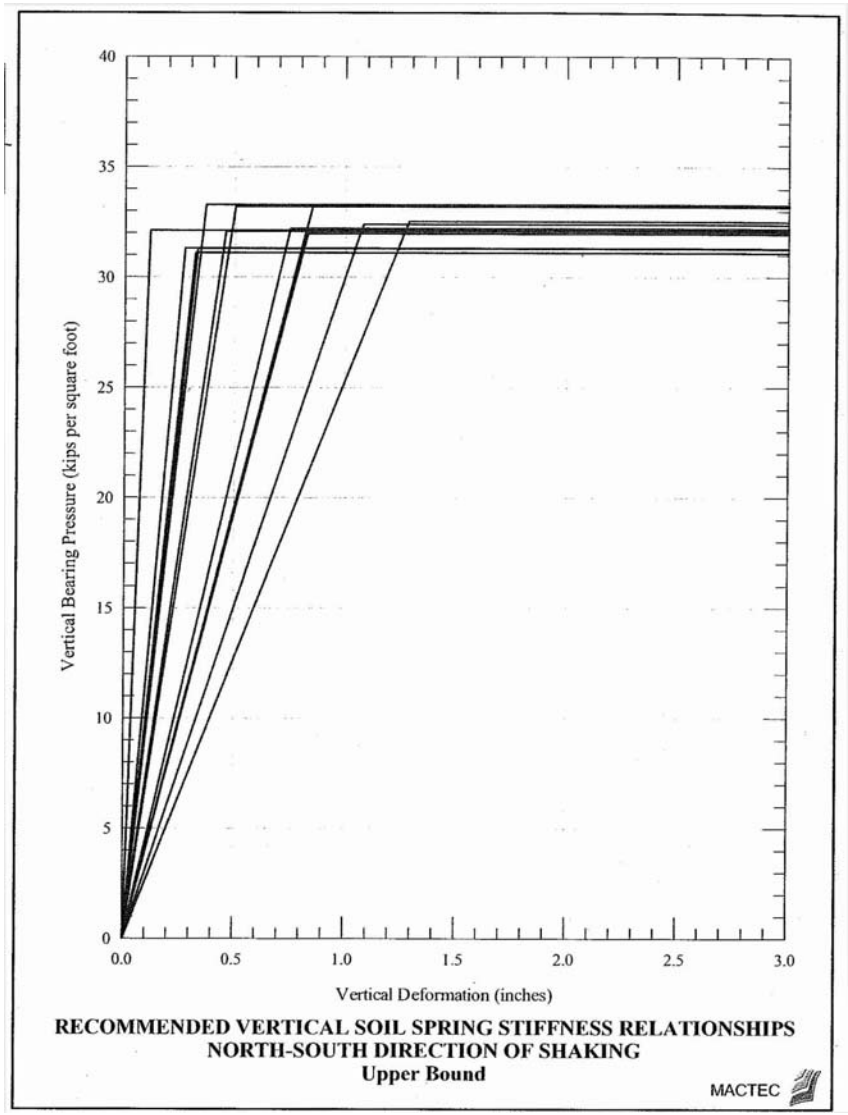
Vertical springs were developed for those portions of footings whose dominant vibration mode was in translation; rotational springs were developed for those portions of footings whose dominant vibration mode was rotational.

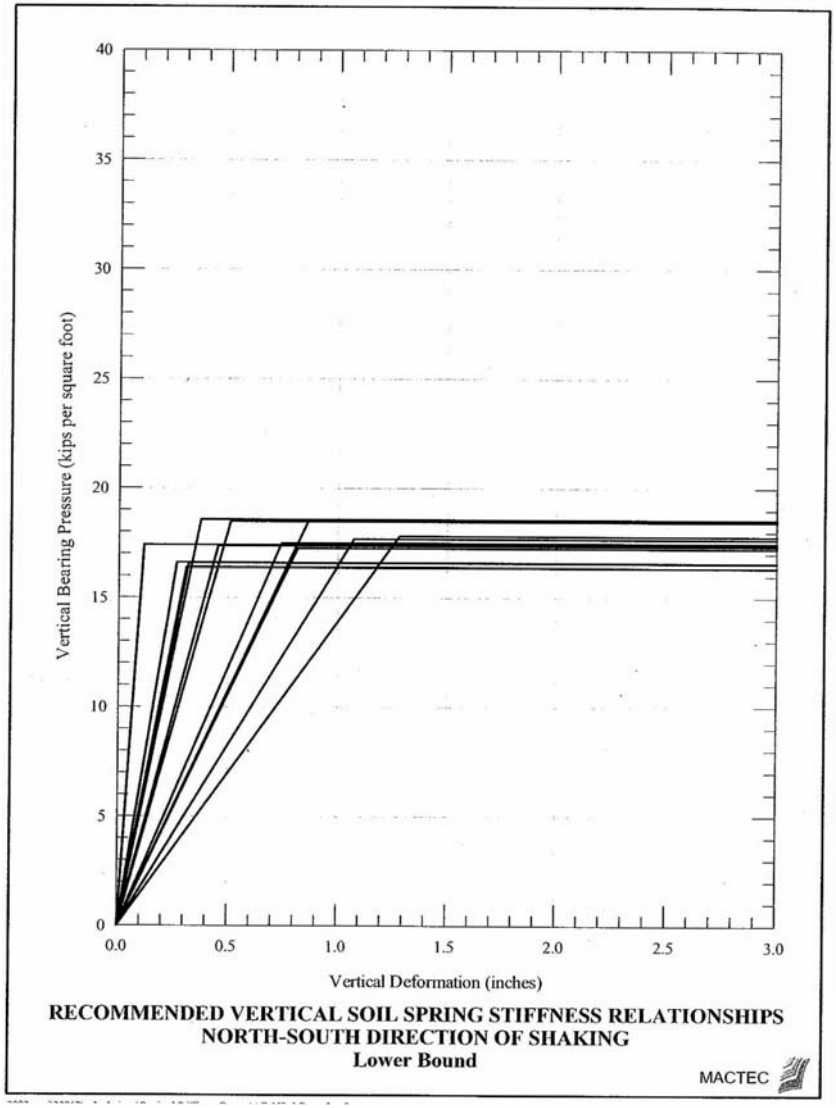
Example – Vertical Spring Stiffnesses – East-West





Example – Vertical Spring Stiffnesses – North-South





Lessons Learned

- We have to get out of our comfort zone and venture into new uncharted waters
- We really don't know what the consequences are of being "conservative"
- We really need to work together
- This is an iterative process requiring close coordination between structural and geotechnical engineers

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References

- American Society of Civil Engineers, Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Prepared for the Federal Emergency Management Agency, FEMA 356, November 2000.
- Stewart et al., Documentation and Analysis of Field Case Histories of Seismic Compression during the 1994 Northridge, California, Earthquake. PEER Report 2002/09, October 2002. (Chapter 7)