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NHI Course No. 13063

Seismic Bridge Design Applications

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Part Two

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16. Abstract

Seismic Bridge Design Applications, Parts One and Two, contains the material used in two one-day national satellite seminars broadcast from the University of Maryland to provide seismic design application instruction. Mr. Robert Mast and Dr. Lee Marsh of BERGER/ABAM Engineers, Inc., were the instructors and developed the course materials. Part One includes seven sessions covering basic seismic principles, one complete seismic analysis and design example, modeling guidelines, multimodal analysis, and column design features. Part Two includes "homework problems" assigned after the first seminar as well as specific topics requested by participants of the first seminar.

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Seismic Design of Bridges
Seminar No. 2 – Outline

Session No.	Topic	Reference Example
1	Practice Problem No. 1 Spread Footings	Concrete Box Girder Bridge (Design Example No. 1)
2	Abutments	
3	Practice Problem No. 2 Conceptual Design Steel Superstructure Issues	Steel Plate Girder Bridge (Design Example No. 2)
4	Skew Structure Issues Elastomeric Bearings	
5	Curved Structure Issues Piles	Curved Box Girder Bridge (Design Example No. 6)

Seismic Design of Bridges
Seminar No. 2 – Outline (continued)

Session No.	Topic	Reference Example
6	Drilled Shafts	Curved Box Girder Bridge (Design Example No. 6)
	Pile Bents	Pile Bent Bridge (Design Example No. 7)
	Joint Design	Other Topics
7	Existing Bridge Assessment and Retrofit	
	Questions and Answers	

Session 1
Concrete Box Girder Bridge Example

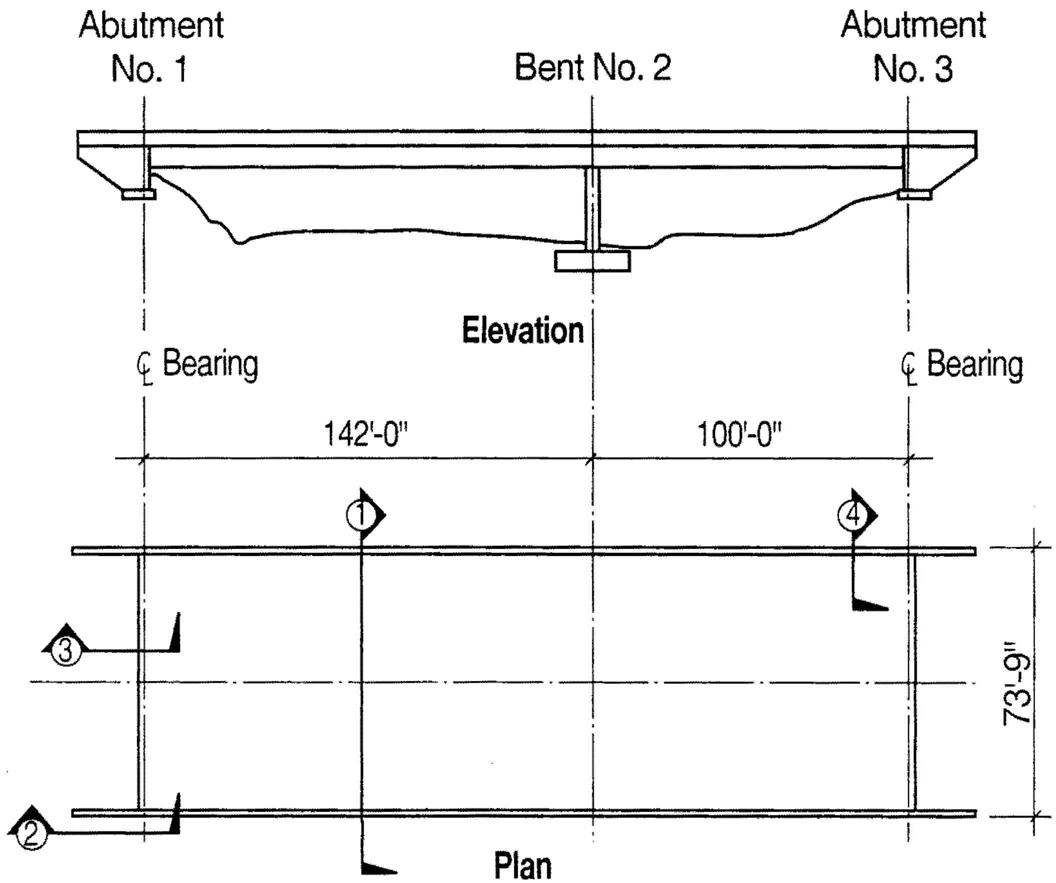
Session 1

- Practice Problem No. 1
- Spread Footings

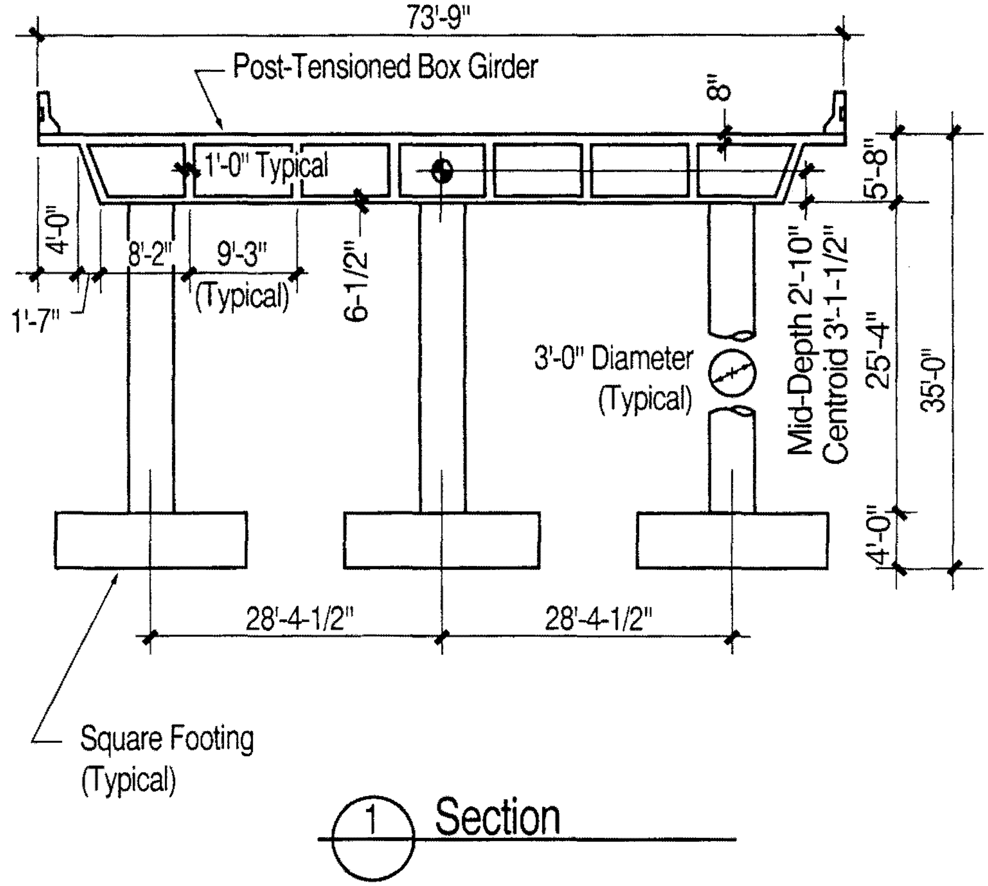
Session 2

- Abutments

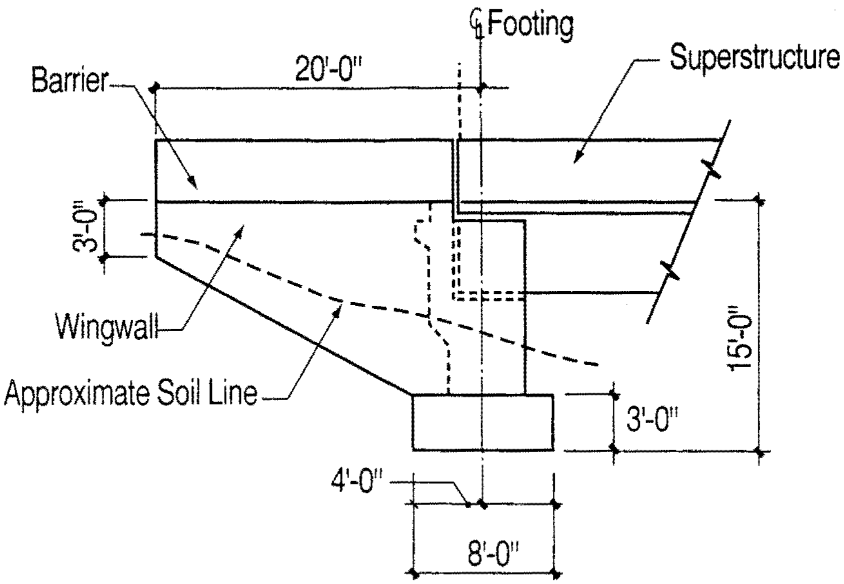
Bridge Layout / Plan and Elevation



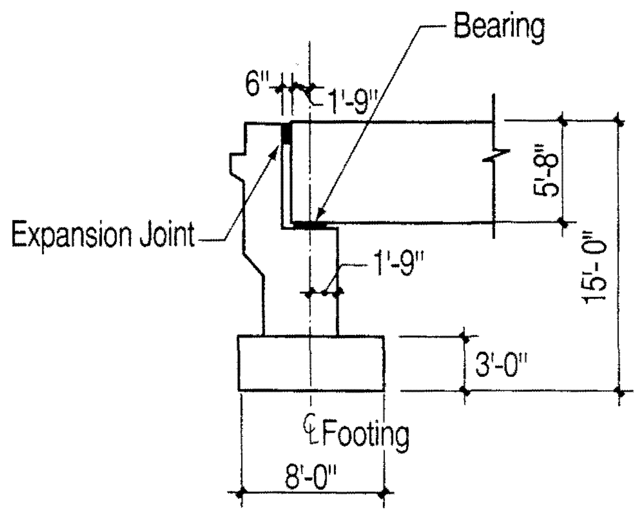
Layout / Preliminary Bent Details



Bridge Layout / Abutment Details

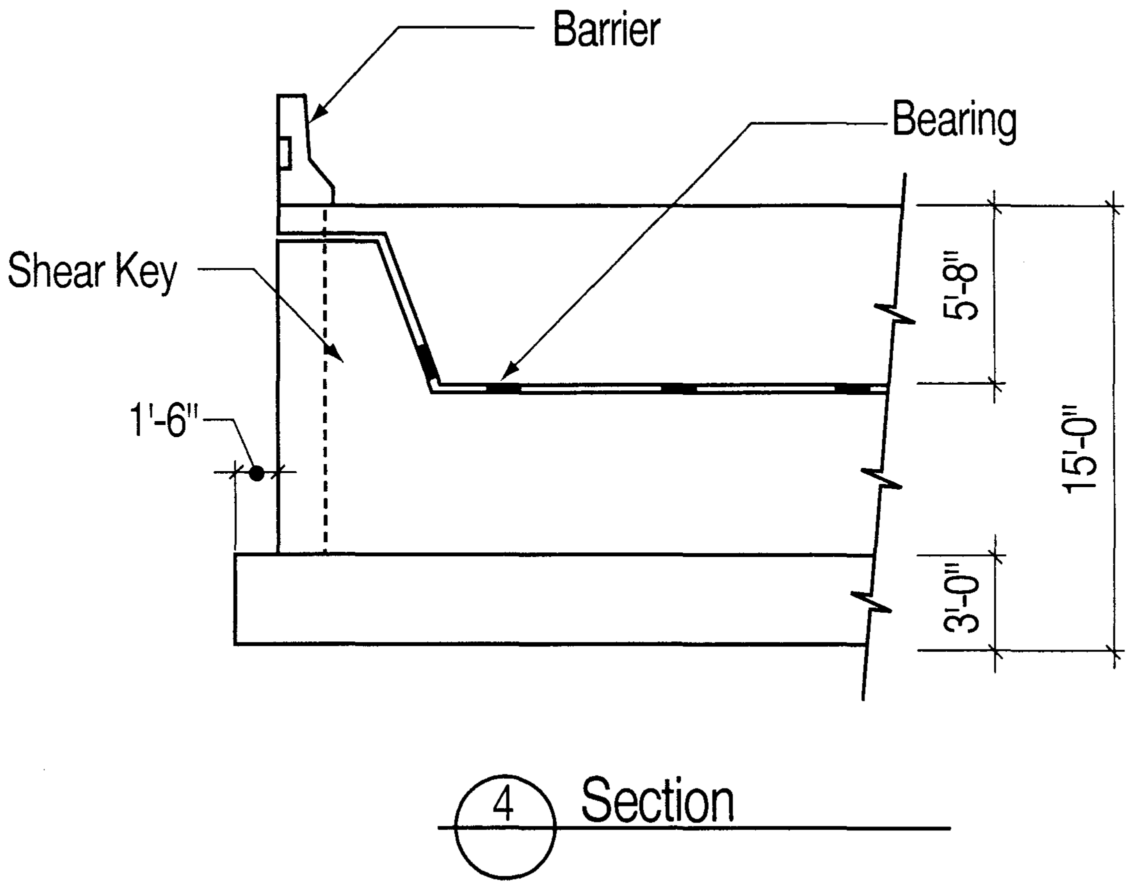


2 Section



3 Section

Layout / Shear Key at Abutments



Session 1
Required / Practice Problem No. 1

- Calculate the Longitudinal Period
- Calculate the Longitudinal Forces and Displacements
- Design the Column Reinforcement
- Size Column Footing
- Assess the Effects of Plastic Hinging

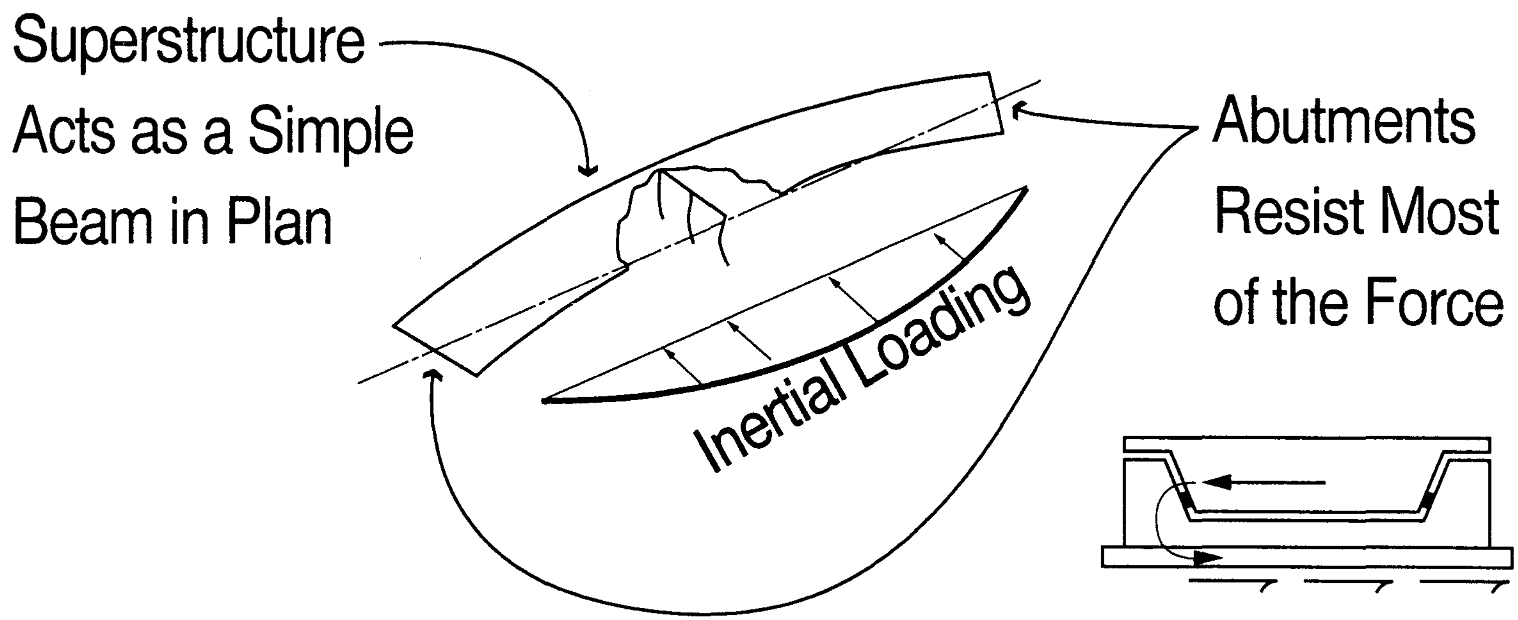
Basic Data for Bridge

- Acceleration Coefficient, A = 0.15g
- Seismic Performance Category, SPC = B
- Soil — 250 ft Deep Glacial Sand and Gravel

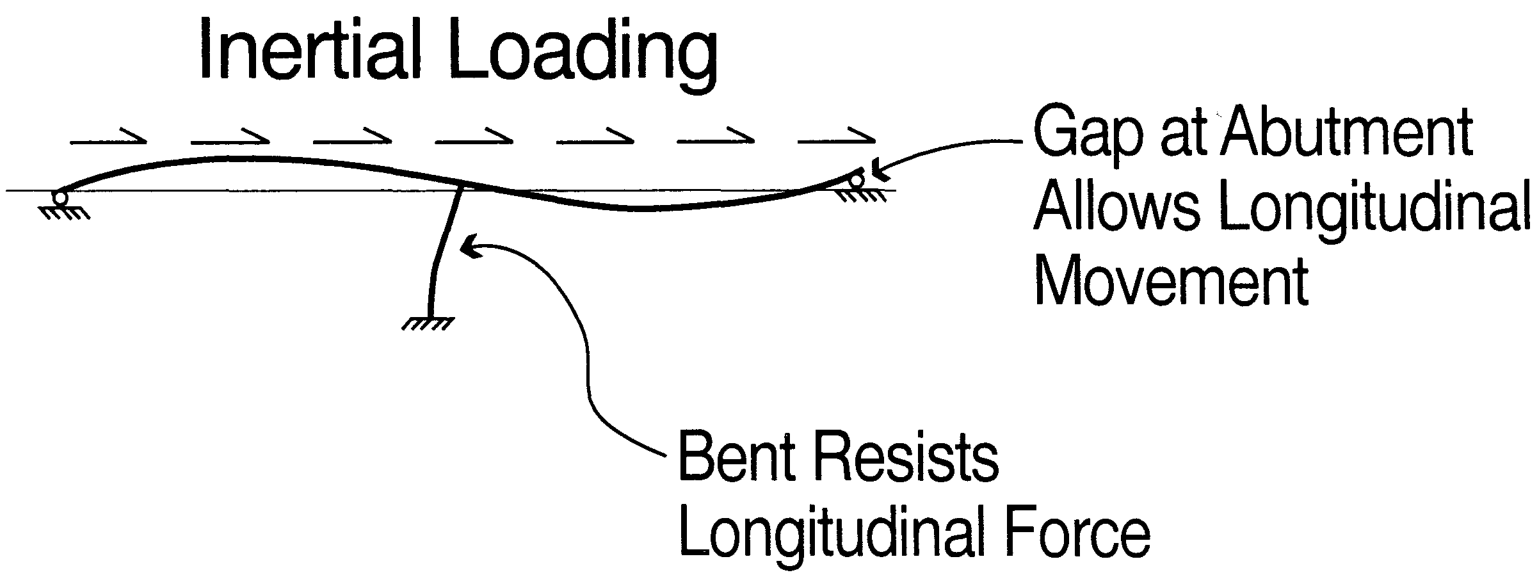
  $S = 1.2$

  $f_{ult} = 24$ ksf

Transverse Lateral Load Behavior

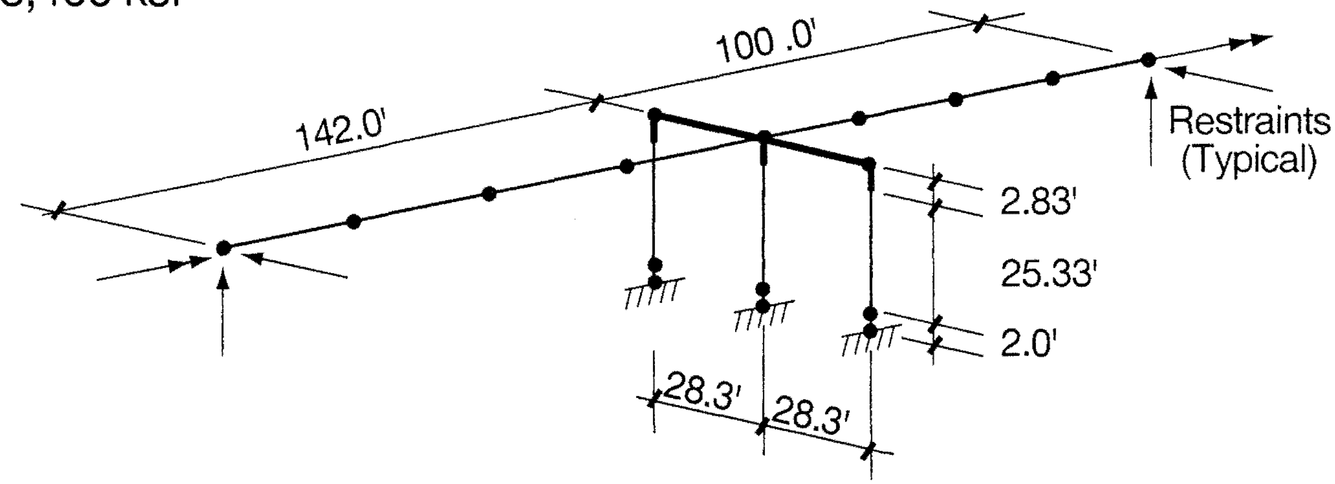


Longitudinal Lateral Load Behavior



Analytical Model and Properties

E = 518,400 ksf



Superstructure

$A = 120\ ft^2$

$I_{str} = 51{,}000\ ft^4$

$I_{weak} = 575\ ft^4$

Capbeam

$A = 25\ ft^2$

$I_{str} = I_{weak} = 10^7\ ft^4$

Column

$A = 7.07\ ft^2$

$I = 3.98\ ft^4$

Longitudinal Period

Stiffness
(Bent Only)

$$K = 3\left(\frac{12EI}{H^3}\right) = 3\left(\frac{12(518{,}400)3.98}{(25.33 + 2.0)^3}\right)$$

$$K = 3639 \text{ kip/ft}$$

Weight

$$W = 4842 \text{ kip}$$

Period

$$T = 2\pi\sqrt{\frac{4842}{32.2\ (3639)}} = 1.28 \text{ sec}$$

$$T_{modal} = 1.32 \text{ sec} \quad (3\% \text{ Difference})$$

Longitudinal Shear and Moment

- **Total Base Shear**

$$C_s = \frac{1.2AS}{T^{2/3}} = \frac{1.2(0.15)\,(1.2)}{(1.28)^{2/3}} = 0.183 < 0.375 = 2.5A$$

$$V_{base} = C_s W = 0.183\,(4842) = 886 \text{ kip}$$

Assumes All Mass Moves Equally

- **Column Forces**

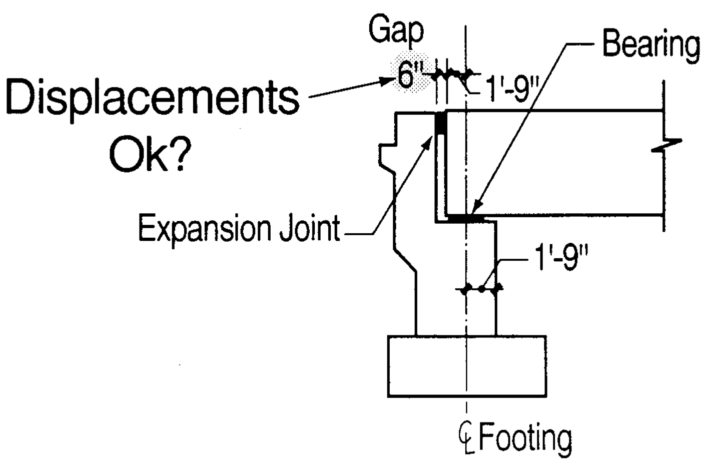
$$V_{col} = \frac{V_{base}}{3} = \frac{886}{3} = 295 \text{ kip vs. } V_{modal} = 288 \text{ kip}$$

$$M_{col} = V_{col}\left(\frac{H}{2}\right) = 295\left(\frac{27.33}{2}\right) = 4031 \text{ kip ft vs. } M_{modal} = 3856 \text{ kip ft}$$

Displacement Calculations

$$\Delta = \frac{V_{base}}{K} = \frac{886}{3639} = 0.24 \text{ ft (2.9 in.)} \quad \text{Gross Properties}$$

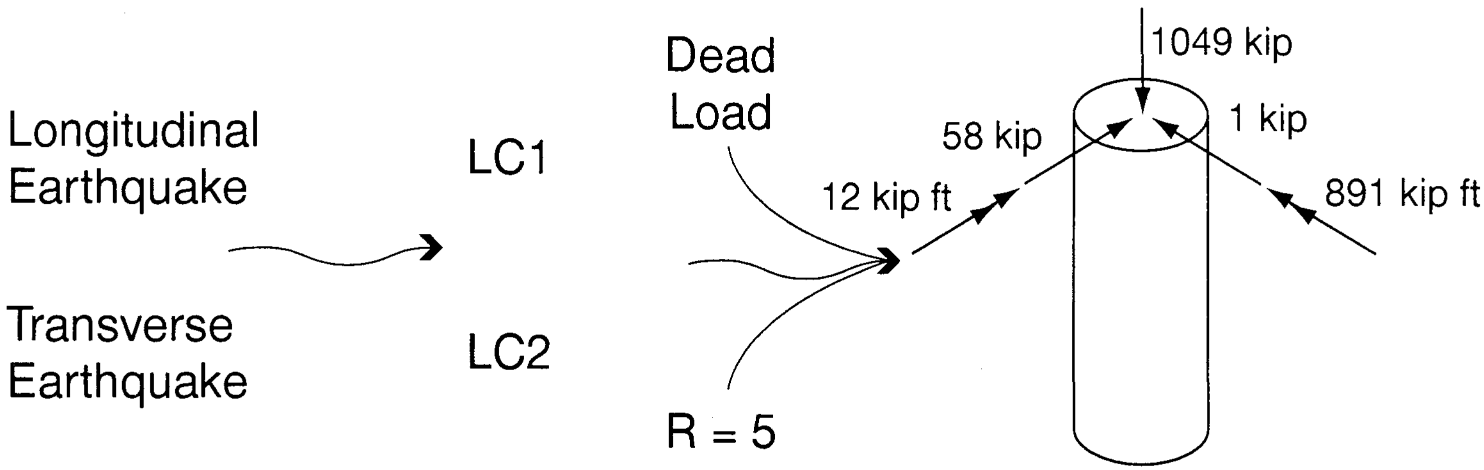
$$\Delta_{l/2} = \frac{0.145(4842)}{1820} = 0.39 \text{ ft (4.6 in.)} \quad \text{Effective / Fixed Base}$$



- Potential for Joint Damage
- Add Footing Flexibility
- More Later

Column Design Forces

- **Outboard Column**



M_{result} = 891 kip ft
V_{result} = 58 kip

Column Flexural Design

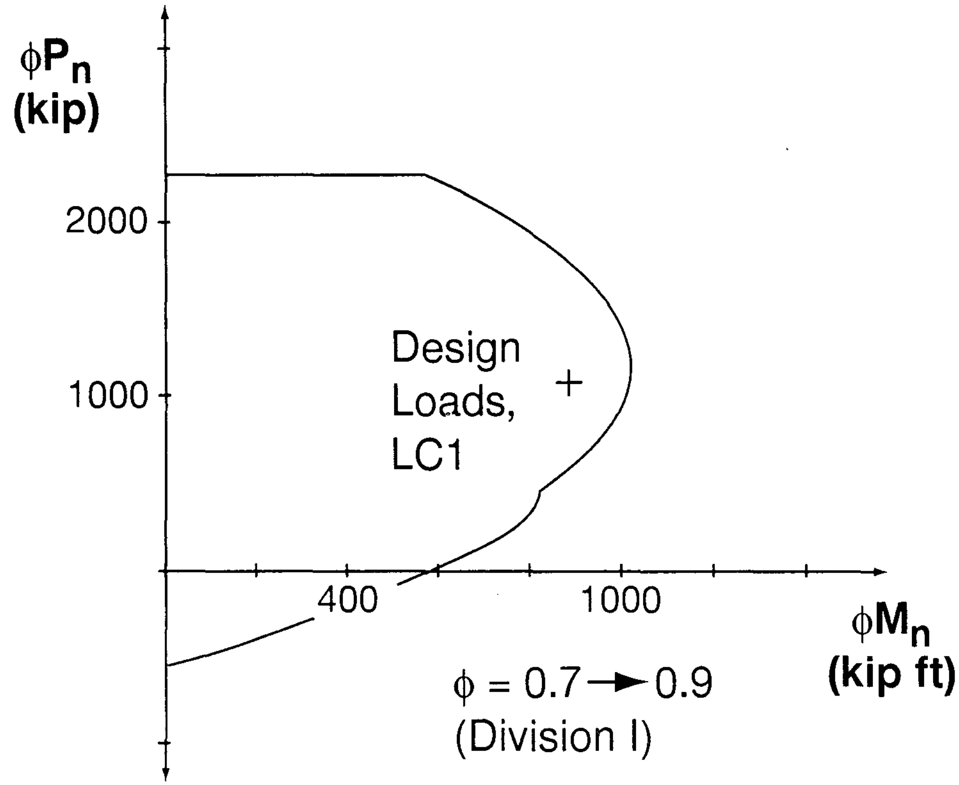
Try:

8 #10 Bars

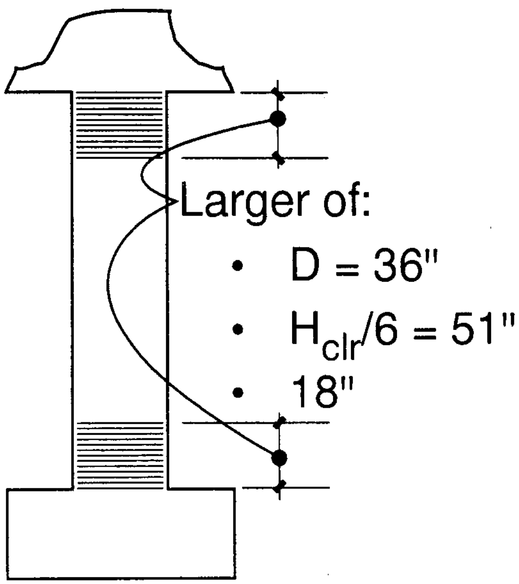
$\rho_g = 1.00\%$

$f_y = 60$ ksi

$f'_c = 4$ ksi



Hinge Zone Confinement



$$\rho_s = 0.45\left(\frac{A_g}{A_{core}} - 1\right)\frac{f'_c}{f_{yh}} = 0.008$$

Minimum:

$$\rho_s \geq 0.12\ \frac{f'_c}{f_{yh}} = 0.008$$

Try $A_{sp} = 0.31\ \text{in}^2$ (#5)

$$s = \frac{4\,A_{sp}\,d_s}{\rho_s\,d_{core}^{\;2}} = \frac{4\,(0.31)(32 - 0.625)}{0.008(32)^2} = 4.75''$$

Use #5 @ 4.5 in. for 60 in.

Shear Strength

SPC B — Shear Strength Same as Division I

$V_u = 58 \text{ kip} \qquad \phi V_c = (0.85)\ \dfrac{2\sqrt{4000}}{1000}\ 36(28) = 109 \text{ kip}$

$$\text{Use } A_{v_{min}} = \frac{50(36)12}{60{,}000} = 0.36 \text{ in}^2$$

Use #5 @ 12 in. $\qquad V_s = 2(0.31)\dfrac{28}{12}\ 60 = 87 \text{ kip}$

$\phi V_n = 109 + 0.85(87) = 183 \text{ kip}$

Footing Design Forces

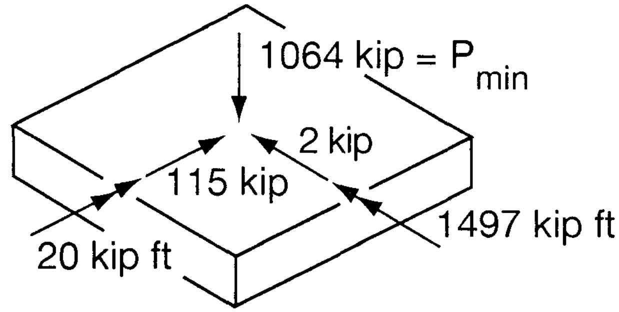
- **Outboard Column**

Longitudinal Earthquake → LC1

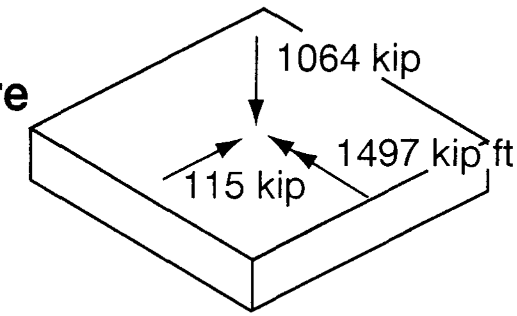
Transverse Earthquake → LC2

Dead Load

$$R = \frac{5}{2} = 2.5$$

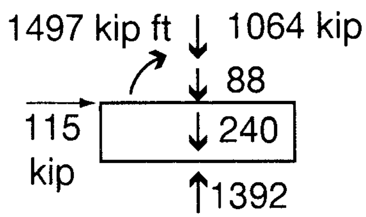


Resultant Forces Essentially Are



Footing Size

- **If B = L = 20 ft**



$$P = P_{D + LC1/R} + P_{footing} + P_{soil}$$

$$P = 1064 \text{ kip} + 240 \text{ kip} + 88 \text{ kip} = 1392 \text{ kip}$$

$$M = 1497 + 115(4) = 1957 \text{ kip ft}$$

$$e = \frac{M}{P} = \frac{1957}{1392} = 1.4 \text{ ft} << \frac{L}{3} = \frac{20}{3} = 6.7 \text{ ft}$$

$\therefore$ 1/2 Uplift
Will Not
Control

- **If B = L = 15 ft** (Gravity Loads Control)

$$P = 1255 \text{ kip}$$

$$e = \frac{1957}{1255} = 1.6 \text{ ft} < \frac{15}{3} = 5 \text{ ft}$$

$$q = 9.5 \text{ ksf} < 24 \text{ ksf}$$

Use 15 ft
Square Footing

Check the Effects of Plastic Hinging

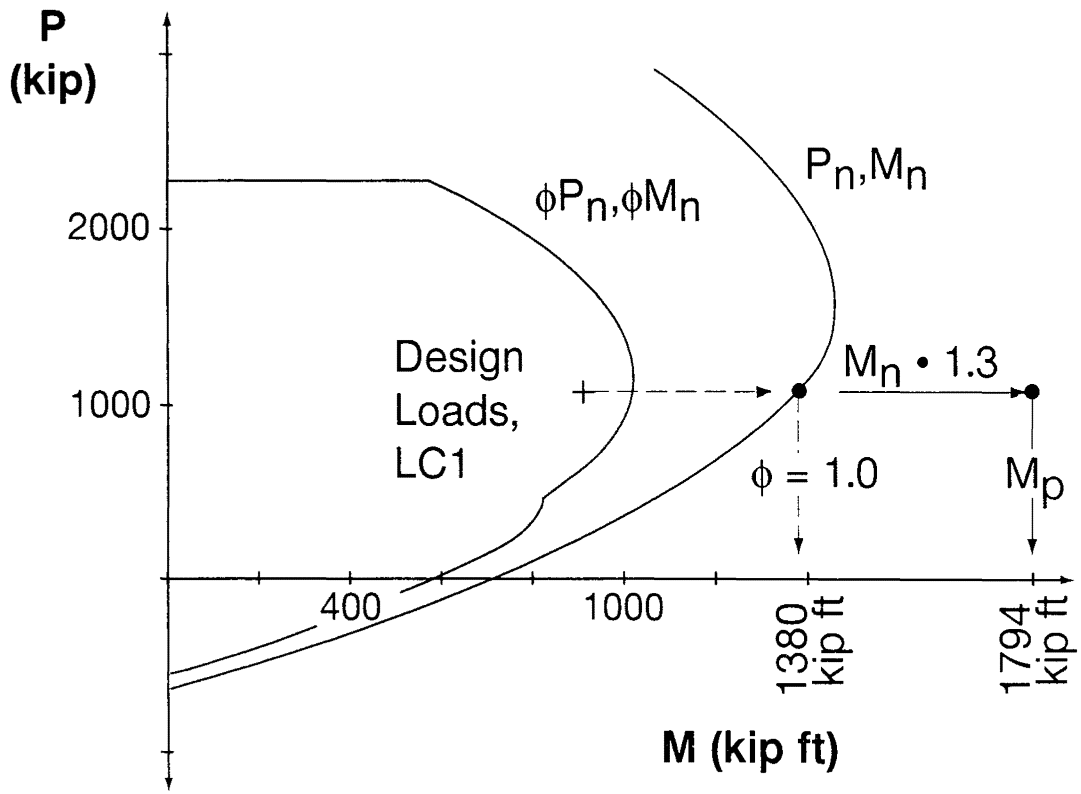
Not Required in SPC B

Column Nominal and Overstrength Properties

8 #10 Bars

$f_y = 60$ ksi

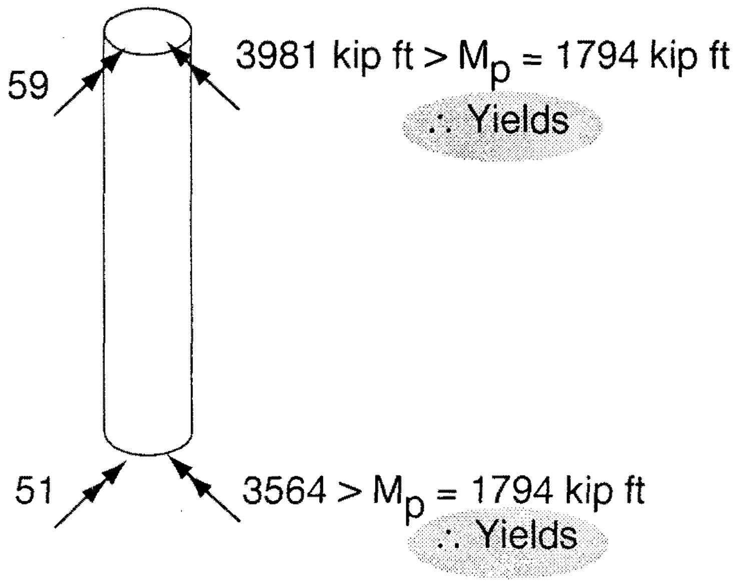
$f'_c = 4$ ksi



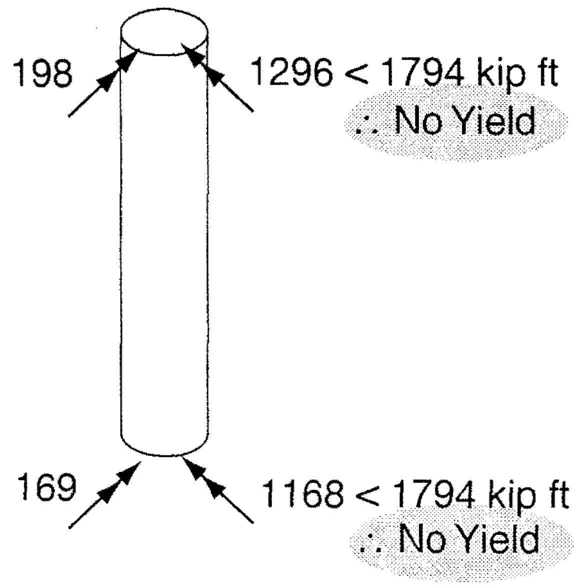
Will Column Develop Plastic Hinge?

Outboard Column

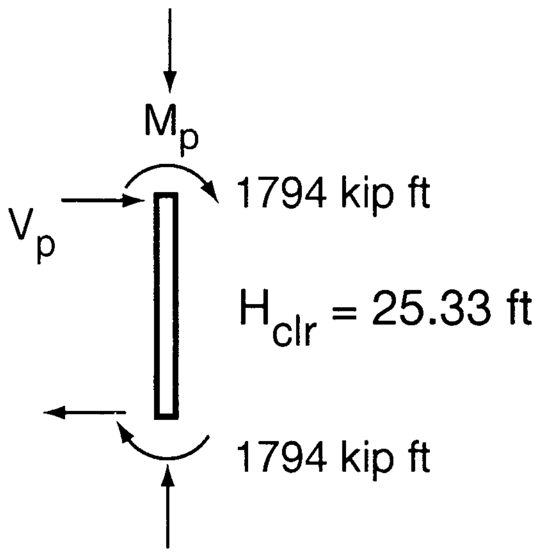
Elastic Forces LC1 + DL



Elastic Forces LC2 + DL



Maximum Column Shear



$$V_{elastic} = 288 \text{ kip}$$

$$V_p = \frac{2(1794)}{25.33} = 142 \text{ kip}$$

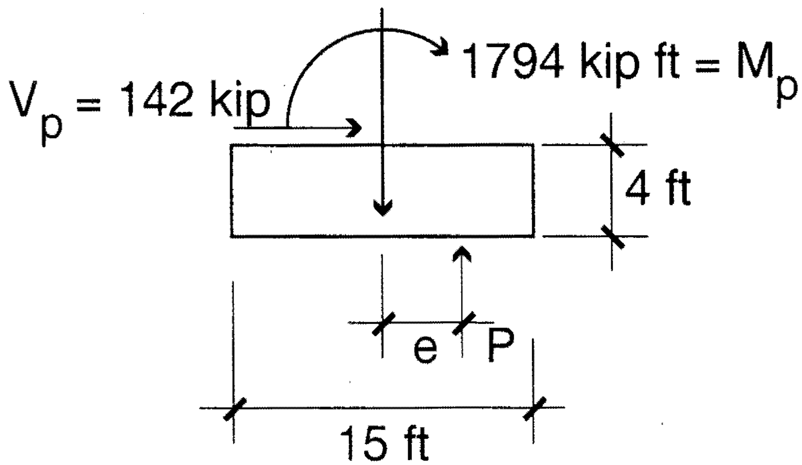
$$\phi V_n = 183 \text{ kip} \quad \therefore \text{ OK}$$

Because We Provided Minimum Steel

Plastic Hinging Effects on Footing

$$e = \frac{M}{P} = \frac{1794 + 142(4)}{1255} = 1.88 \text{ ft} < 5 \text{ ft}$$

and $q = 9.9$ ksf < 24 ksf

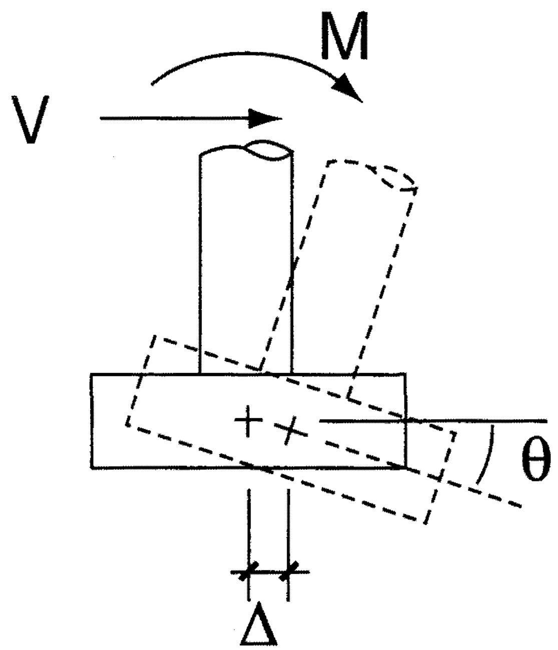


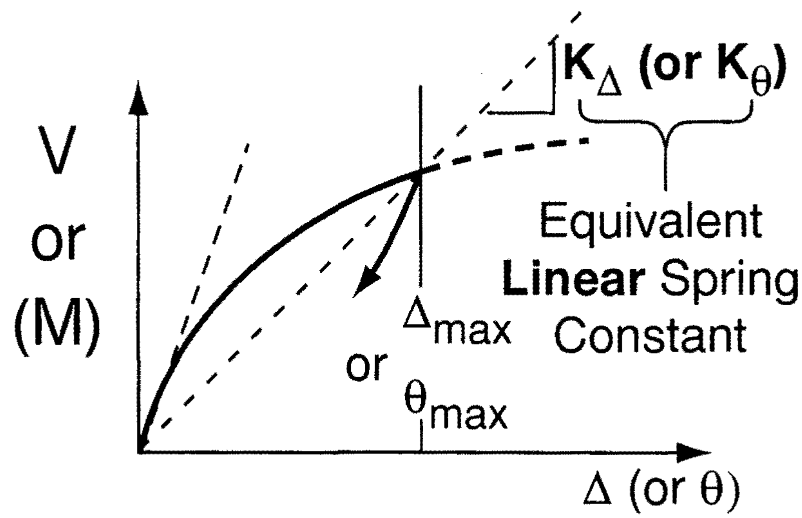
$\therefore$ OK
for Plastic
Hinging

Session 1
Spread Footings

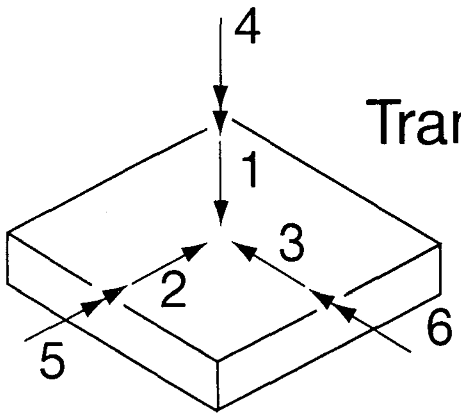
- **Including Flexibility**
- Overturning and Sliding
- Pinned Base Columns

Conceptual Behavior





Degree-of-Freedom / Importance

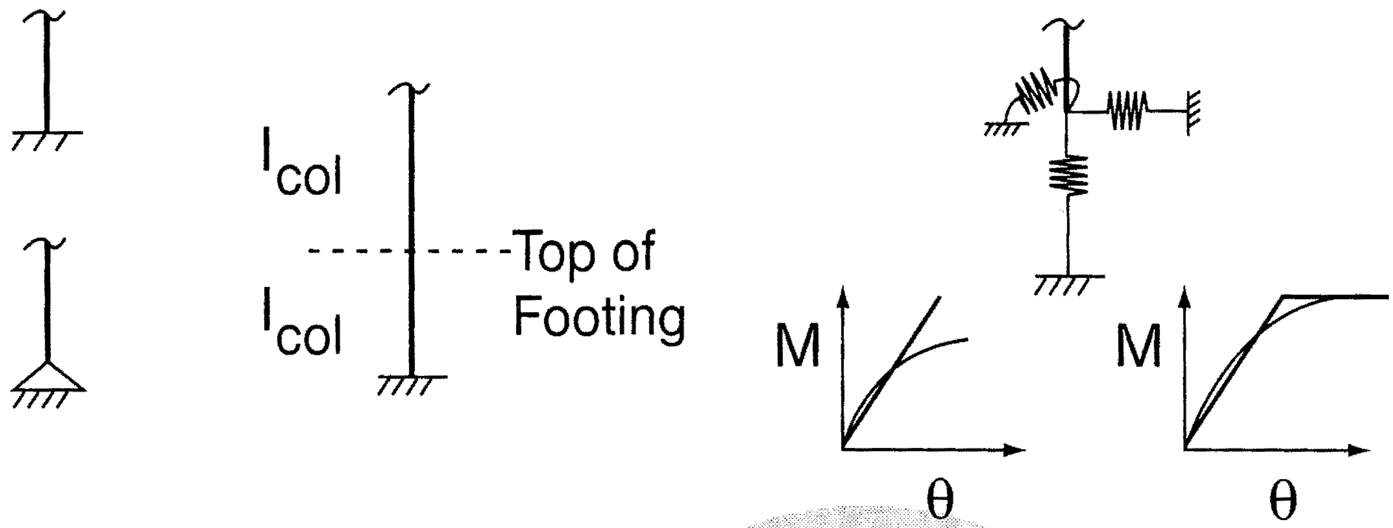


Translation
1. Vertical
2. Lateral
3. Lateral

Rotation
4. Torsion
5. Rocking
6. Rocking

Most Important to Include

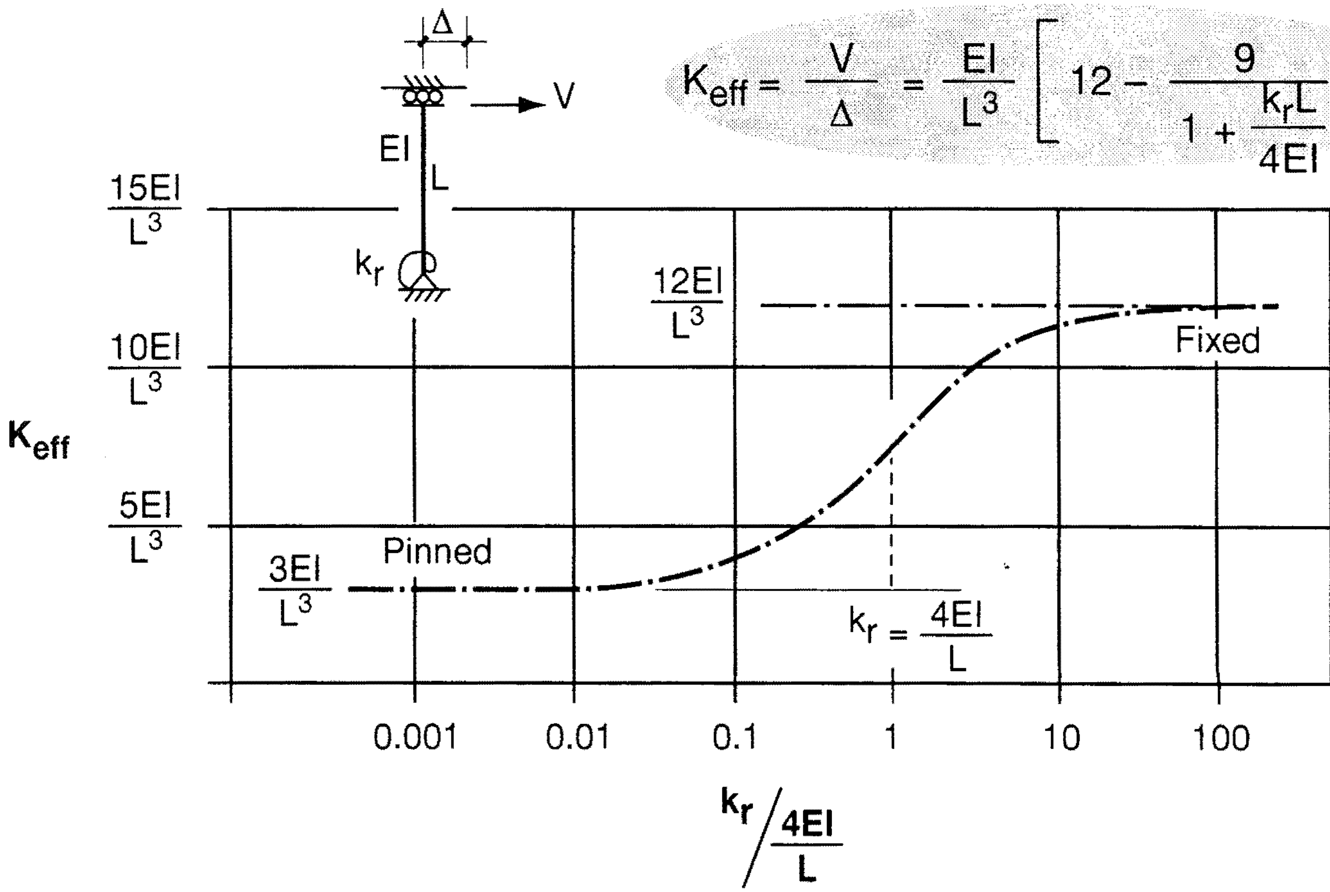
Modeling Foundation Flexibility



Fixed / Free | **Equivalent Column** | **Linear Soil Springs** | **Nonlinear Springs**

Rotational Flexibility / Fixed or Not?

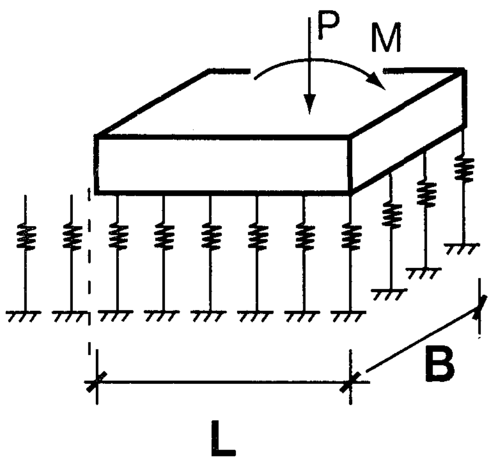
$$K_{eff} = \frac{V}{\Delta} = \frac{EI}{L^3}\left[12 - \frac{9}{1 + \frac{k_r L}{4EI}}\right]$$



Determining Foundation Stiffness

- Elastic Foundation Methods
- 'Elastic Half-Space' Methods

Elastic Foundation Method

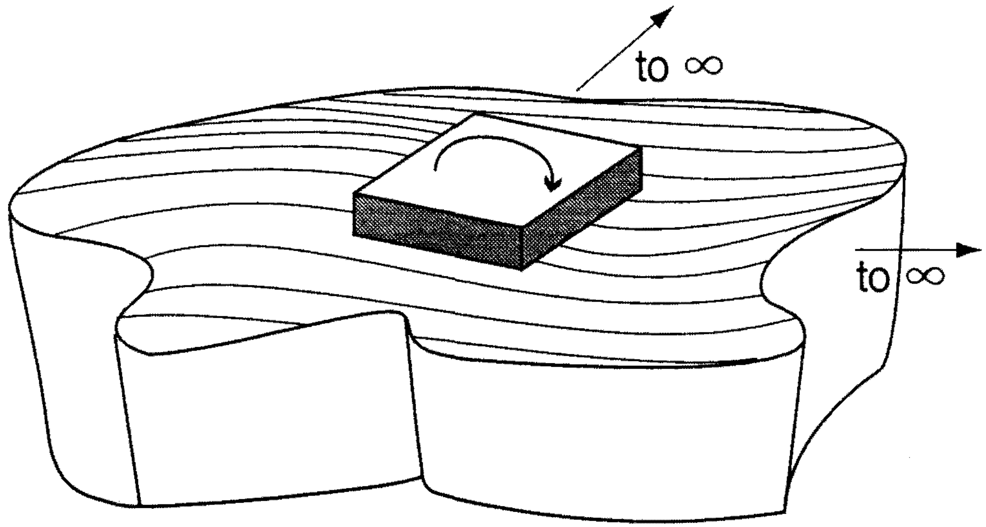


k_S, Subgrade Reaction Coefficient

$$\left(\frac{\text{kip}}{(\text{ft}^2 \text{ of Area})(\text{ft of Deflection})} = \text{kcf} \right)$$

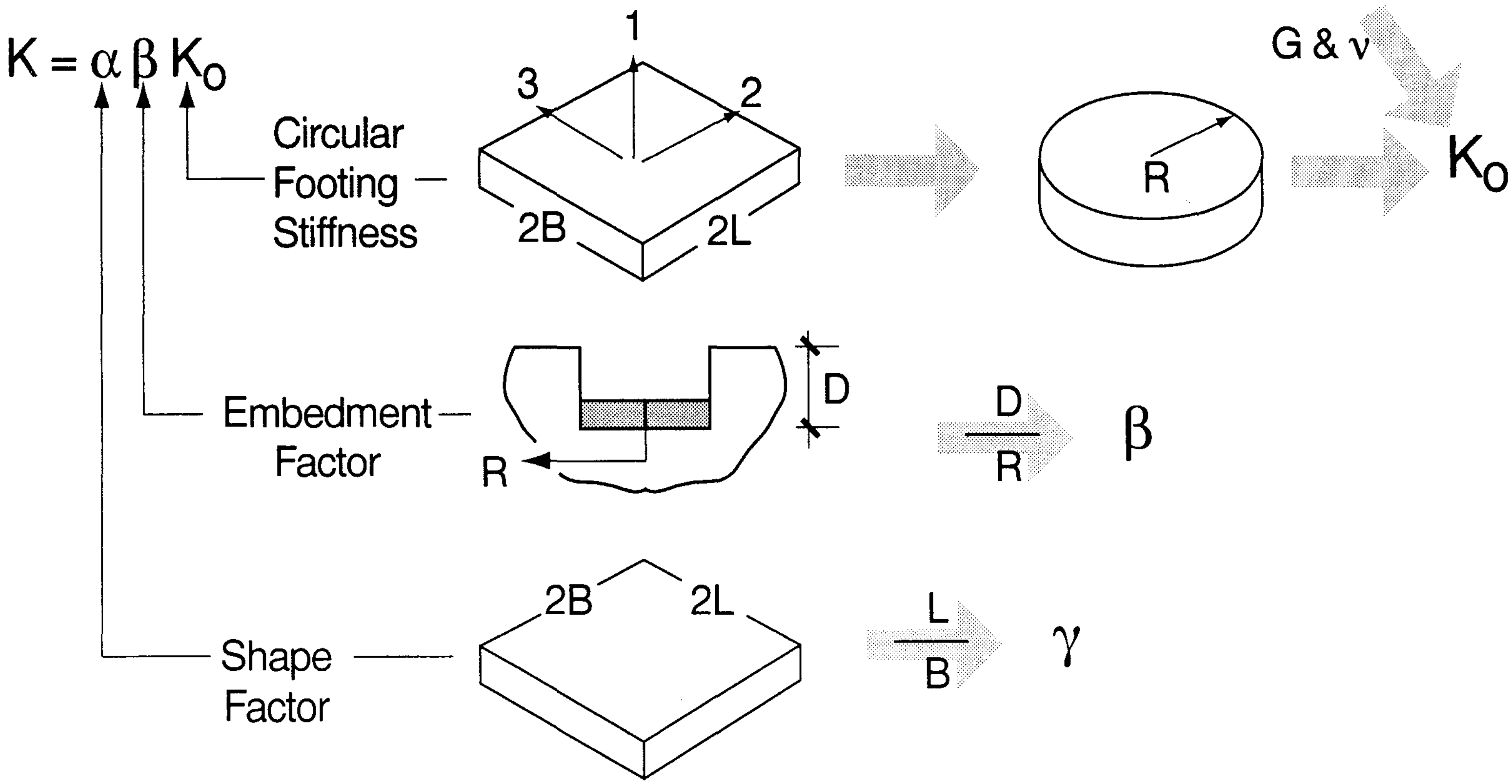
- 'Springs' Are Independent (Winkler Foundation)
- Footing Rigid Relative to Soil
- Rotational Stiffness, $k_r = k_S \dfrac{L^3 B}{12} \quad \dfrac{\text{kip ft}}{\text{rad}}$

Half-Space Method



- Footing (Rigid) Bonded to Elastic Half-Space Medium
- Must Use Theory of Elasticity Methods to Determine K's
  (Standard Non-Dimensional Solutions)

Half-Space Method for Spread Footings



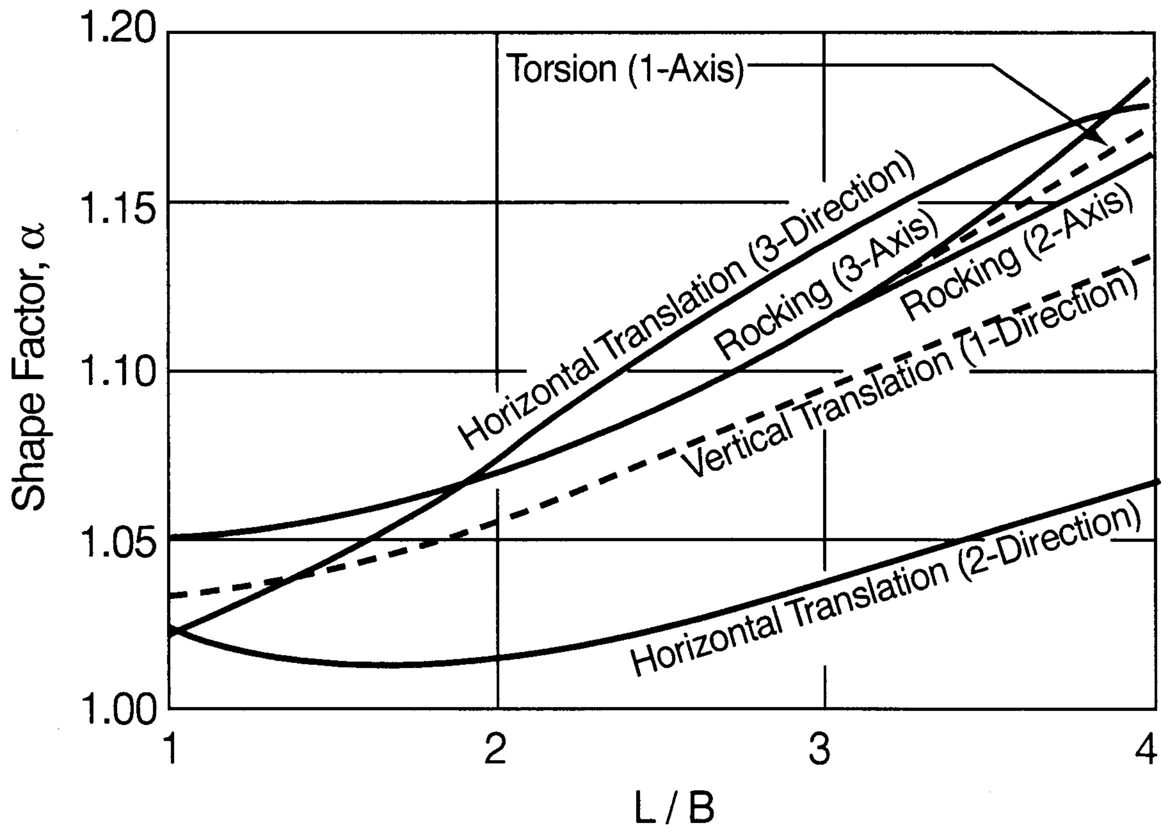
Adapted from : FHWA-IP-87-6

Stiffness of Circular Surface Footing

Degree of Freedom	Equivalent Radius	Stiffness K_0
Vertical Translation	$R_0 = \sqrt{\dfrac{4BL}{\pi}}$	$4GR/1 - \nu$
Lateral Translation (Both)	"	$8GR/2 - \nu$
Torsion Rotation	$R_1 = \left[\dfrac{4BL\,(4B^2 + 4L^2)}{6\pi}\right]^{1/4}$	$16GR^3/3$
Rocking About 2	$R_2 = \left[\dfrac{(2B)^3\,(2L)}{3\pi}\right]^{1/4}$	$8GR^3/3(1 - \nu)$
Rocking About 3	$R_3 = \left[\dfrac{(2B)\,(2L)^3}{3\pi}\right]^{1/4}$	"

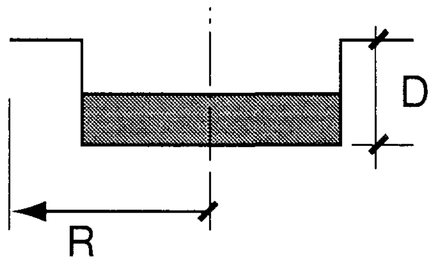
Adapted from: FHWA-IP-87-6

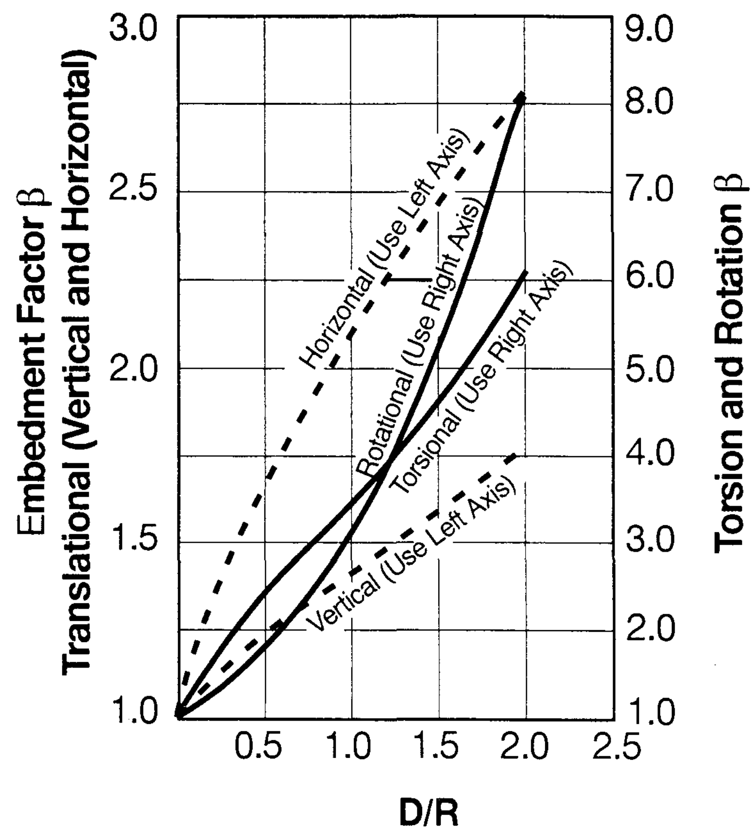
Shape Factor for Rectangular Footing



Adapted from : FHWA-IP-87-6

Embedment Factor





Adapted from : FHWA-IP-87-6

Representative* Soil Properties

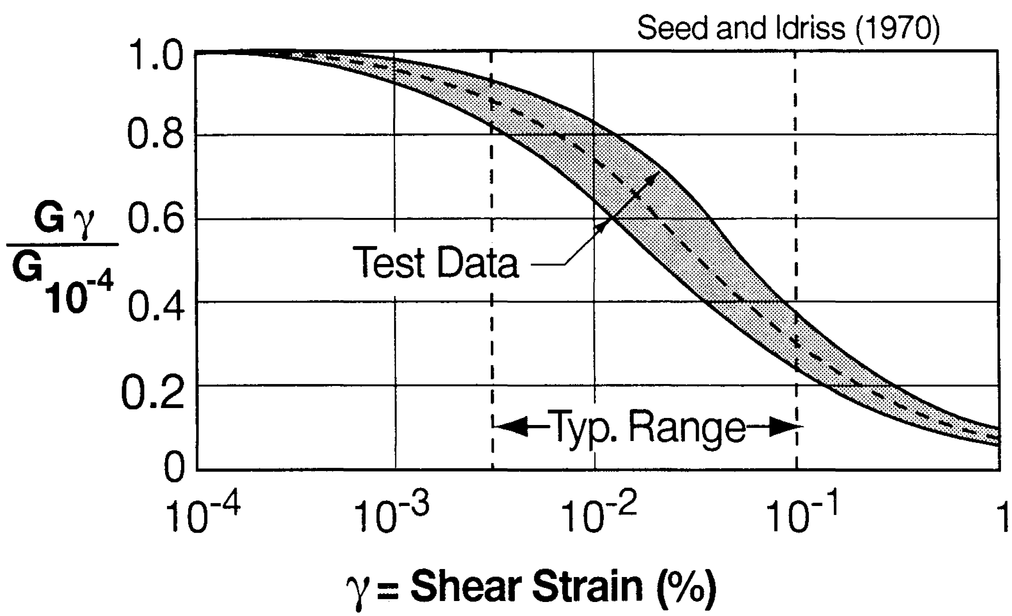
- **Shear Modulus, G**

Material	G(ksi)
Clean dense quartz sand	1.8-3+
Micaceous fine sand	2.3
Berlin sand (*e*=0.53)	2.5-3.5
Loamy sand	1.5
Dense sand-gravel	10+
Wet soft silty clay	1.3-2
Dry soft silty clay	2.5-3
Dry silty clay	5-5
Medium clay	2-4
Sandy clay	2-4

Bowles (1988)

* Consult Your Geotech!

- **Poisson's Ratio** $\nu = 0.3$ Cohesionless
  $\nu = 0.4 - 0.5$ Cohesive
- **Shear Modulus vs. Strain**



Example / Rocking Stiffness / Half-Space

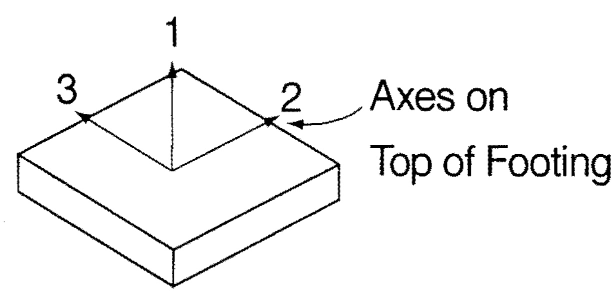
- **Consider Practice Problem No. 1**

Footing: $2B = 2L = 15$ ft $\quad D = 6$ ft

Soil: From Geotechnical Engineer, $G = 400$ ksf $\quad \nu = 0.3$

Rotational Stiffness: $K_{r3} = \alpha \, \beta \, K_0$

(Rocking About Axis 3)



Example / Rocking Stiffness (continued)

- Equivalent Radius, $R_3 = \left[\frac{(15)(15)^3}{3\pi}\right]^{1/4} = 8.56 \text{ ft}$
- Rocking, K_0 $\quad K_0 = \left[\frac{8(400)(8{,}56)^3}{3(1-0.3)}\right] = 955{,}600 \ \frac{\text{kip ft}}{\text{rad}}$
- Shape Factor, α $\quad \frac{L}{B} = 1 \rightarrow \alpha = 1.05$
- Embedment Factor, β $\quad \frac{D}{R} = \frac{6}{8.56} = 0.70 \rightarrow \beta = 2.3$

Example / Rocking Stiffness (continued)

- $K_{r3} = \alpha\beta K_0 = 1.05(2.3)\ 955{,}600 = 2{,}308{,}000\ \dfrac{\text{kip ft}}{\text{rad}}$

- How Important Is This Stiffness on the Lateral Behavior of the Structure?

Column Properties

$E = 518{,}400$ ksf

$I = 3.98\ \text{ft}^4$

$H_{clr} = 25.33$ ft

$$K_{eff} = \frac{EI}{H^3}\left[12 - \frac{9}{1 + \frac{k_r L}{4EI}}\right]$$

$$\frac{K_{\theta 3} H}{4EI} = 7.08 \Rightarrow K_{eff} = 10.9\frac{EI}{H^3}$$

vs. 12! ∴ Essentially Fixed

Example / Footing Rocking – Practice No. 1

- Effective Longitudinal Stiffness Including Rocking

$$K_{eff} = 3\left(10.9\,\frac{EI}{H^3}\right) = 4146 \text{ kip/ft}$$

- Previously in Practice No. 1 K = 3639 kip/ft (Top Half of Footing Included with I_{col} to Approximate Footing Flexibility)

- New Results

  T = 1.20 sec (vs. 1.28 sec)

  $C_s = 0.192$

  V = 928 kip

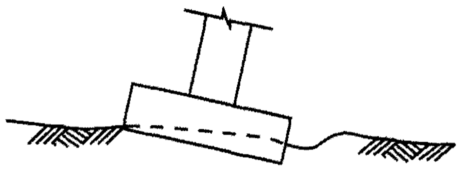
  $$\Delta_{long} = 2.7 \text{ in} \quad \text{vs.} \begin{cases} 2.9 \text{ in with } I_g \\ 4.6 \text{ in with } I_g/2 \end{cases}$$

Session 1
Spread Footings

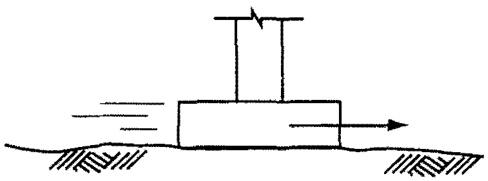
- Including Flexibility
- **Overturning and Sliding**
- Pinned Base Columns

Spread Footing Failure Modes

• Soil Failure



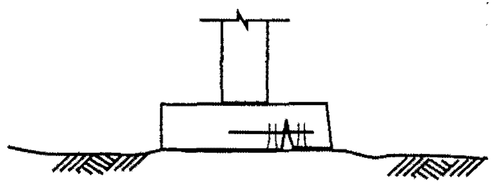
Soil Bearing Failure
(Overturning)



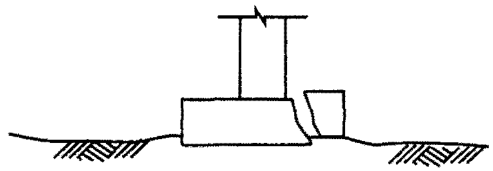
Sliding Failure

• Footing Failure

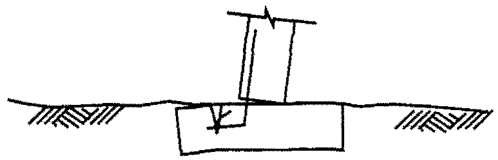
(All Types Aggravated by Large Overturning)



Flexural Yielding of Reinforcing



Concrete Shear Failure



Anchorage Failure

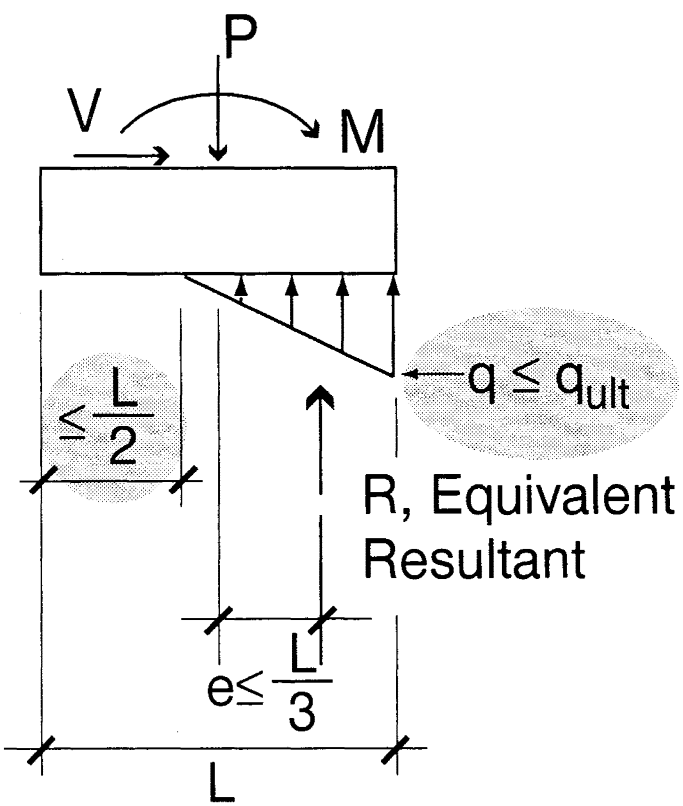
Overturning

Division I-A, Articles 6.4.2(B) and 7.4.2(B)

"Because of the dynamic cyclic nature of seismic loading, the ultimate capacity of the foundation medium should be used ..."

"Transient foundation uplift or rocking involving separation ... up to one-half of ... pile group or ... contact area is permitted ... provided that ... soils are not susceptible to loss of strength ..."

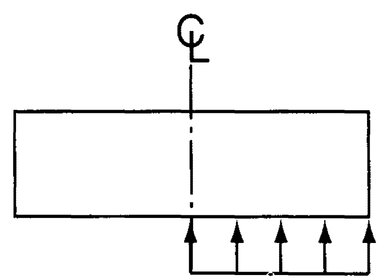
Overturning Comparisons



- Triangular Stress Distribution

  Recommended for Now

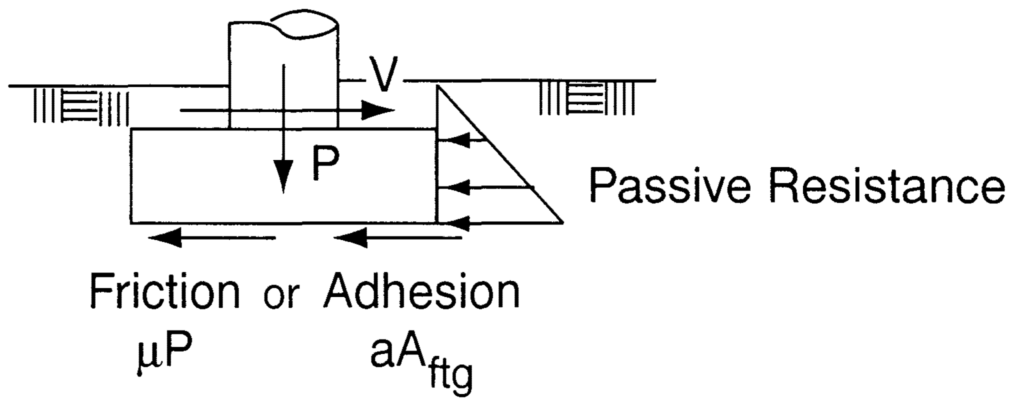
- Rectangular Stress Distribution



Under Development,
Better Correlation with
Test Results?
Better for Soft Soils?

Sliding

- Make Comparisons at Impending Sliding Condition
- Neglect Passive Resistance? (Consult Your Geotech)
- If Soil Is Adhesive, Use Larger of Friction or Adhesion
- Consider Jointing Effect in Rock



Representative* Ultimate Values of Coefficient of Friction for Concrete Foundations on Rock / Soil

Material	Relative Density/ Consistency	Coefficient of Friction[1]	Adhesion[1] (PSF)[2]
Clean, Sound Rock[3]	Not Applicable	0.70 - 0.80	—
Clean Gravel, Gravel-Sand Mixtures	Dense to Very Dense Medium Dense	0.55 - 0.70 0.55 - 0.65	—
Clean to Slightly Silty / Clayey Sand with or without Gravel	Dense to Very Dense Medium Dense	0.45 - 0.60 0.45 - 0.55	— —
Silty / Clayey Sand and Sandy Silt with or without Gravel	Dense to Very Dense Medium Dense	0.40 - 0.55 0.35 - 0.50	— —
Siltly Clay and Clayey Silt with or without Sand and Gravel (low plasticity)[4]	Very Stiff to Hard Medium Stiff to Stiff	0.40 - 0.50 0.30 - 0.45	1000 - 1500 500 - 1000

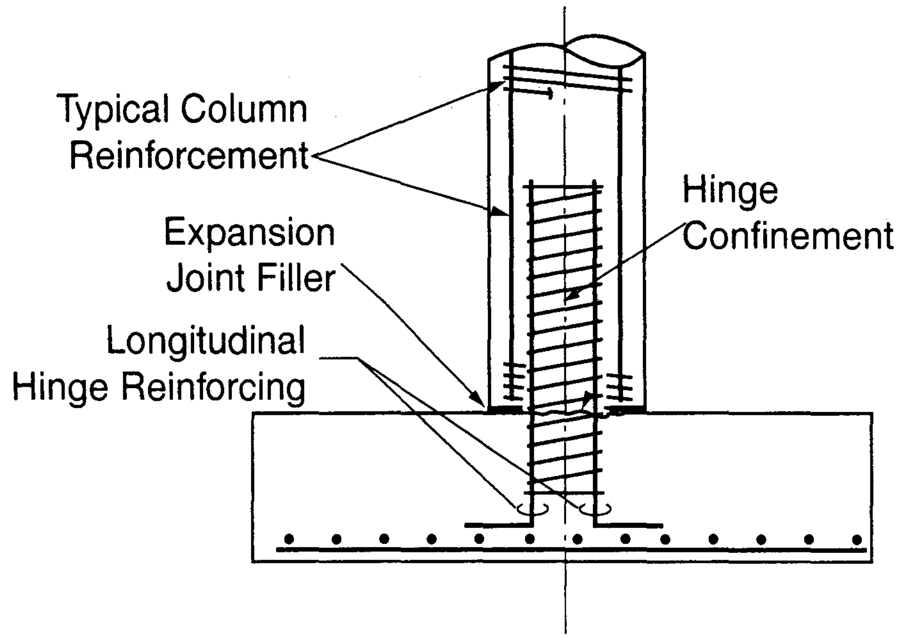
(After Potyondy, 1961; Goh and Donald, 1984; U.S. Department of the Navy, 1986) **For Notes 1 through 4, See Design Example No. 3**

*** Consult Your Geotechnical Engineer**

Session 1
Spread Footings

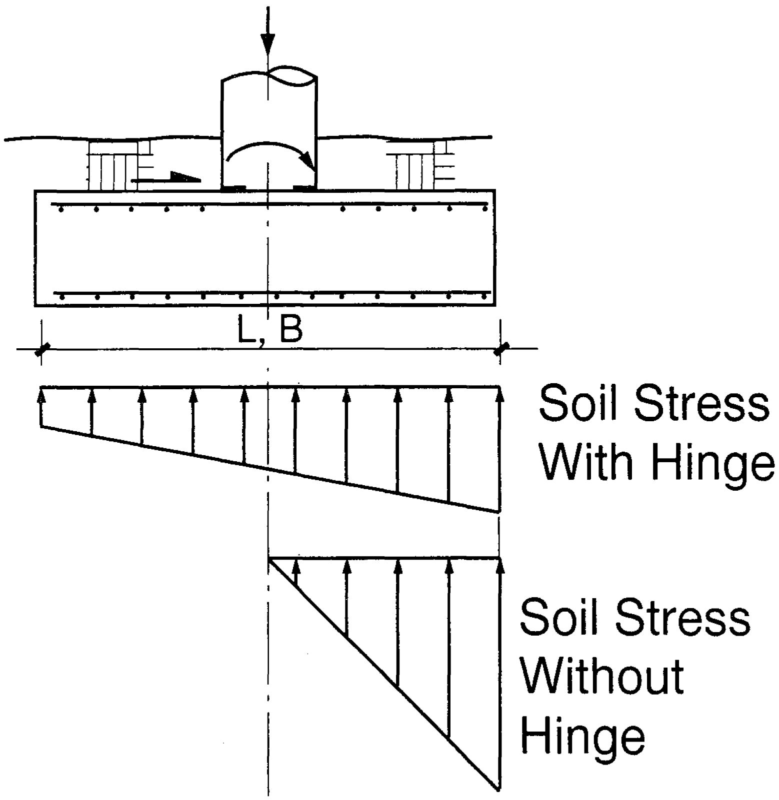
- Including Flexibility
- Overturning and Sliding
- **Pinned Base Columns**

Limiting the Moment Transferred to a Footing



Seismic Hinge Detail

Effects of Limiting Foundation Moments



With a Hinge:

- Soil Contact Stress Lower
- Internal Forces Lower
- Structure More Flexible (Displacements Larger)
- Can Reduce Footing Size
- May Increase Column Size

Design of Pinned Bases

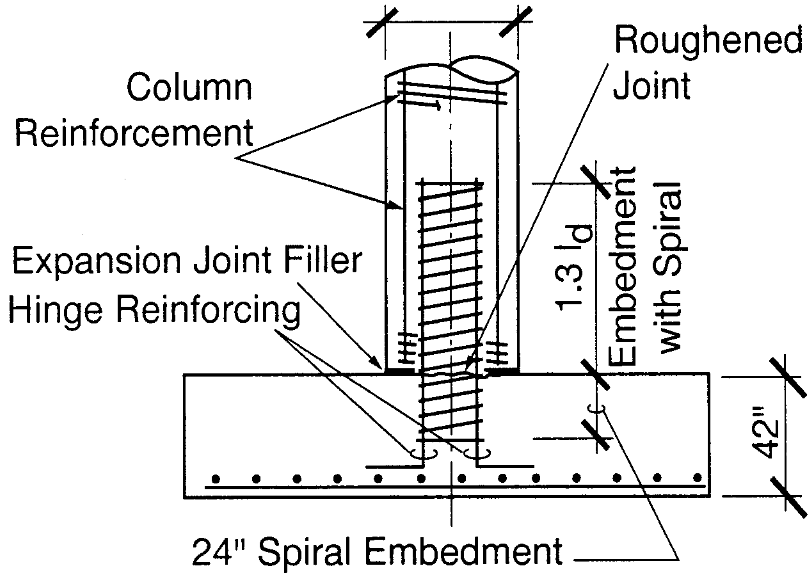
- Use 1/2 in. or More Expansion Joint Filler for Rotation Capacity
- Size Contact Area Using Shear Friction
- Ensure Area Can Carry Group VII Loads Based on

$$\phi P_o = 0.85\phi f'_c(A_g - A_{st}) + A_{st} f_y$$

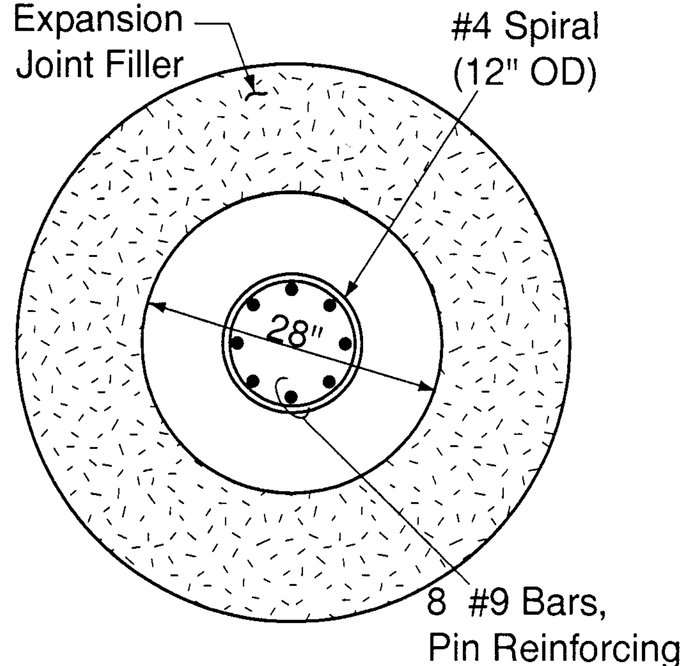
Caltrans (1995)

- Centralize Longitudinal Steel to Minimize Actual Moment
- Develop Longitudinal Steel on Both Sides of Hinge
- Use a Nominal Spiral Over Half the Column Dimension Above and Below Hinge

Example Detailing / 4 ft Diameter Column



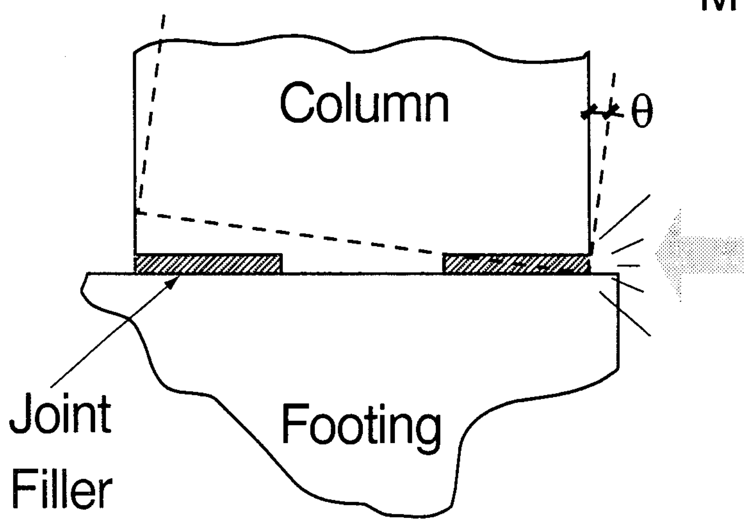
Section Through Column

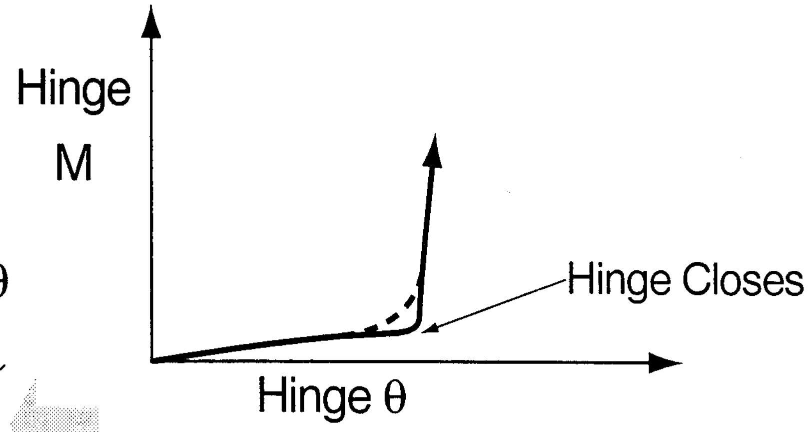


Section Through Hinge

Reference: Design Example No. 4

Limit Behavior / Pinned Base Columns



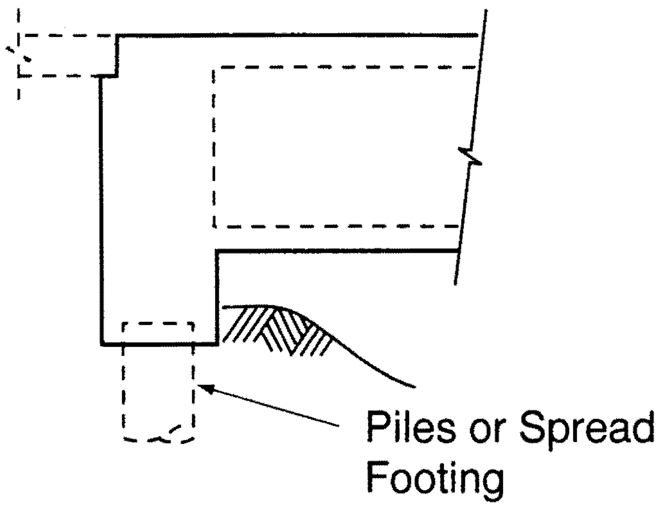


- Check Elastic Hinge Rotations for Closing

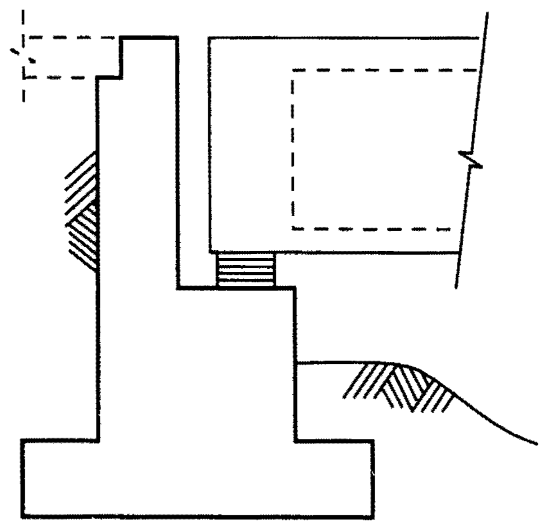
Session 2
Concrete Box Girder Bridge Example
Abutments

- Conceptual Behavior
- Modeling Soil Flexibility
- Nonlinear Effects
- Mononobe-Okabe Analysis
- Design Issues, Force Transfer, and Fuse Elements

Types of Abutments

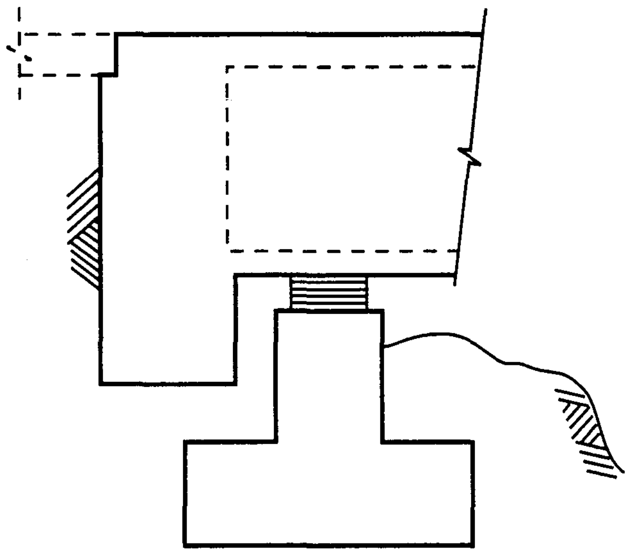


Integral Abutment
(Monolithic)

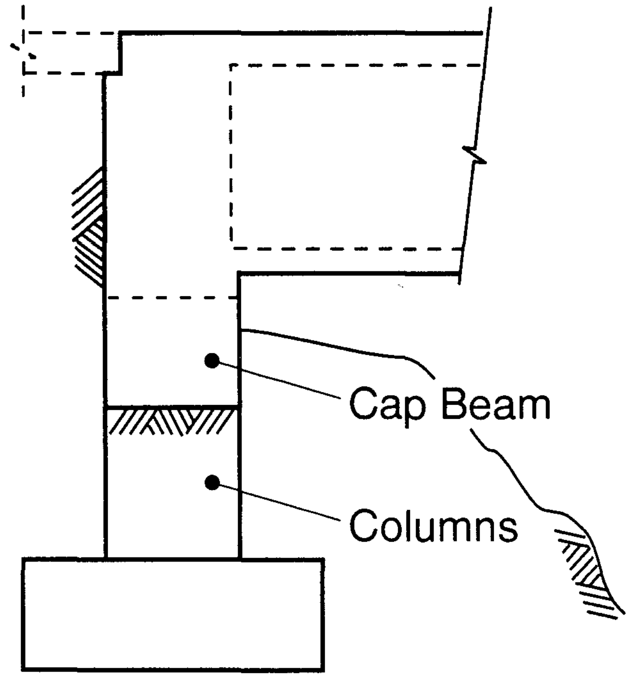


Seat Abutment
(Free-Standing)

Variations of the Integral Abutment



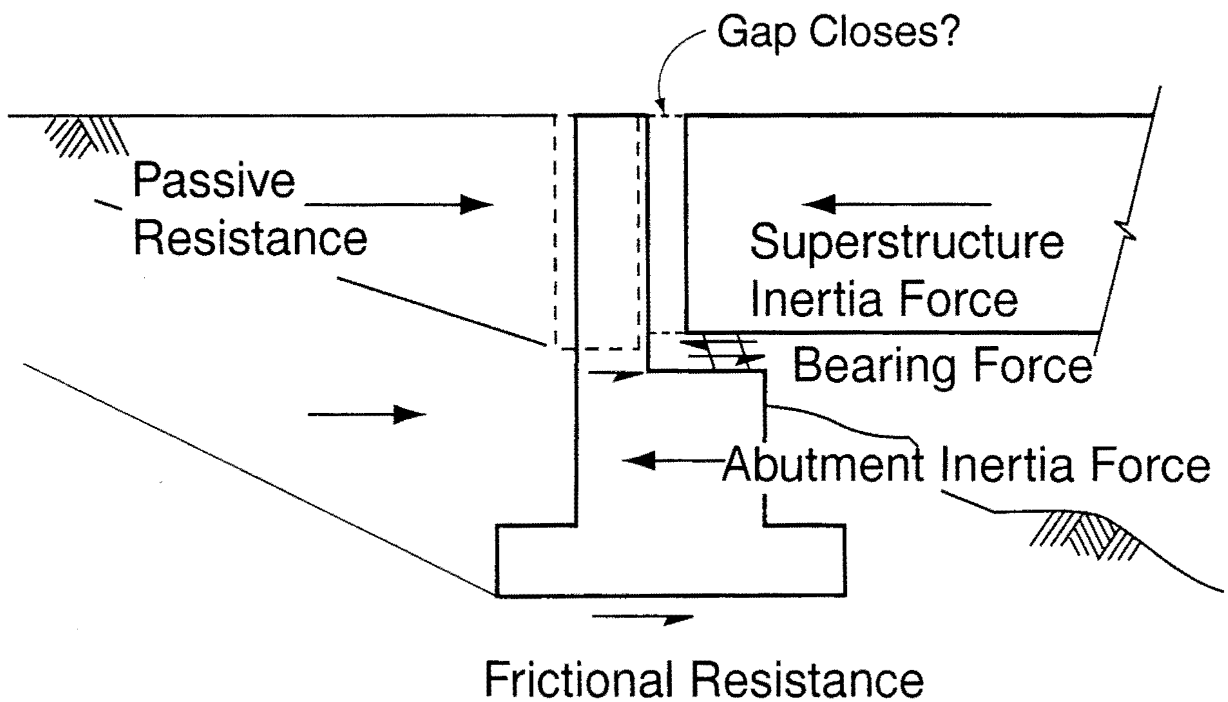
Stub Abutment
(Semi-Integral)



Spill-Through Abutment

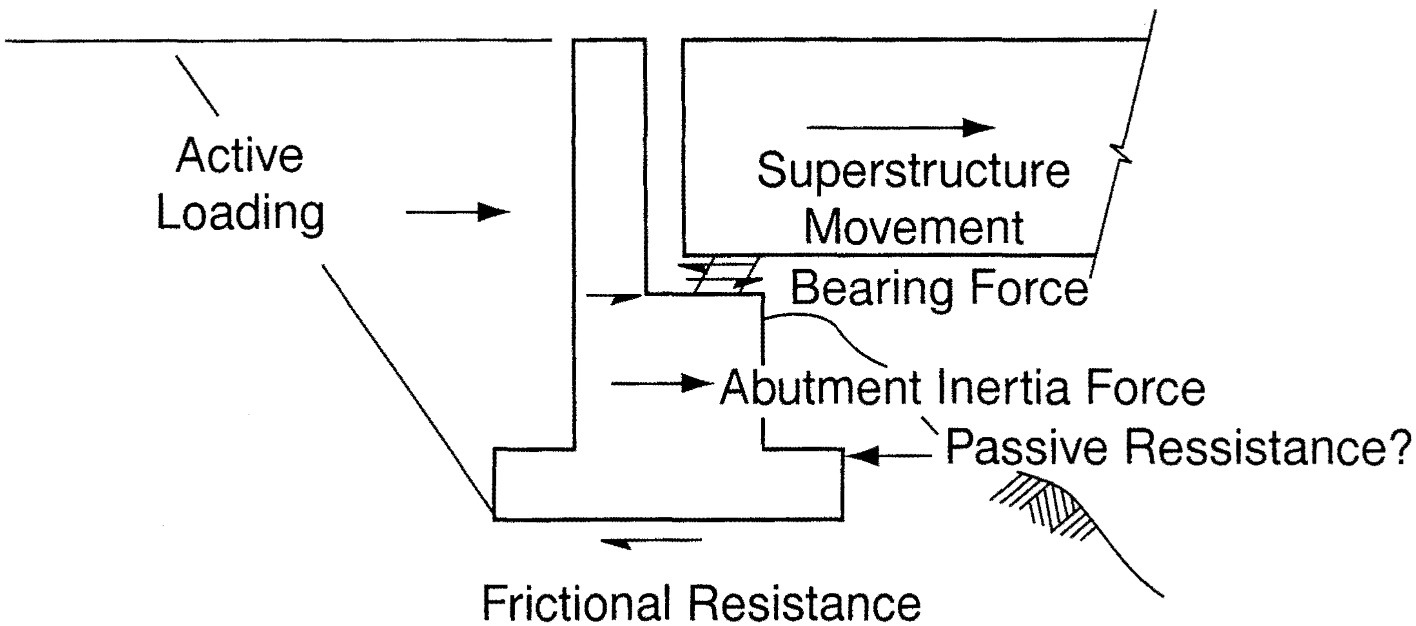
Seat Type / Longitudinal Behavior

Superstructure Moves **Toward** Backfill

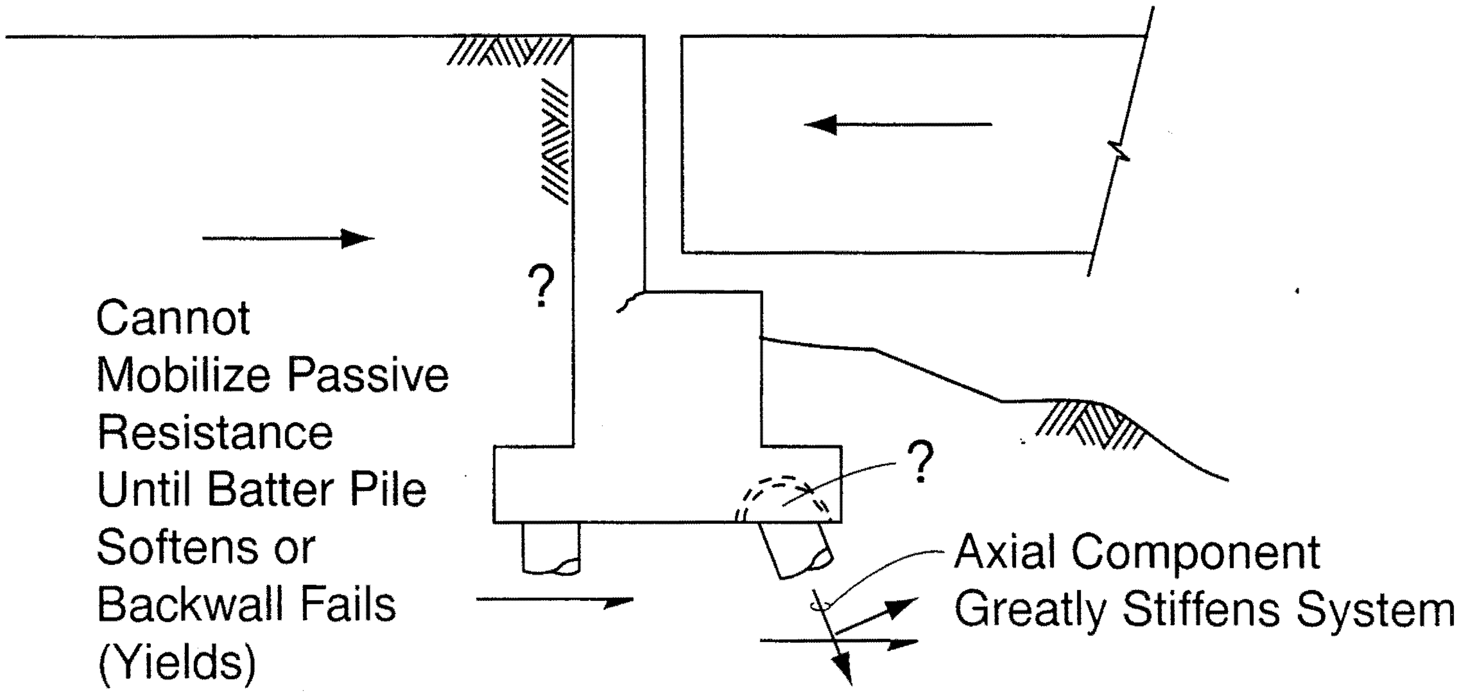


Seat Type / Longitudinal Behavior (continued)

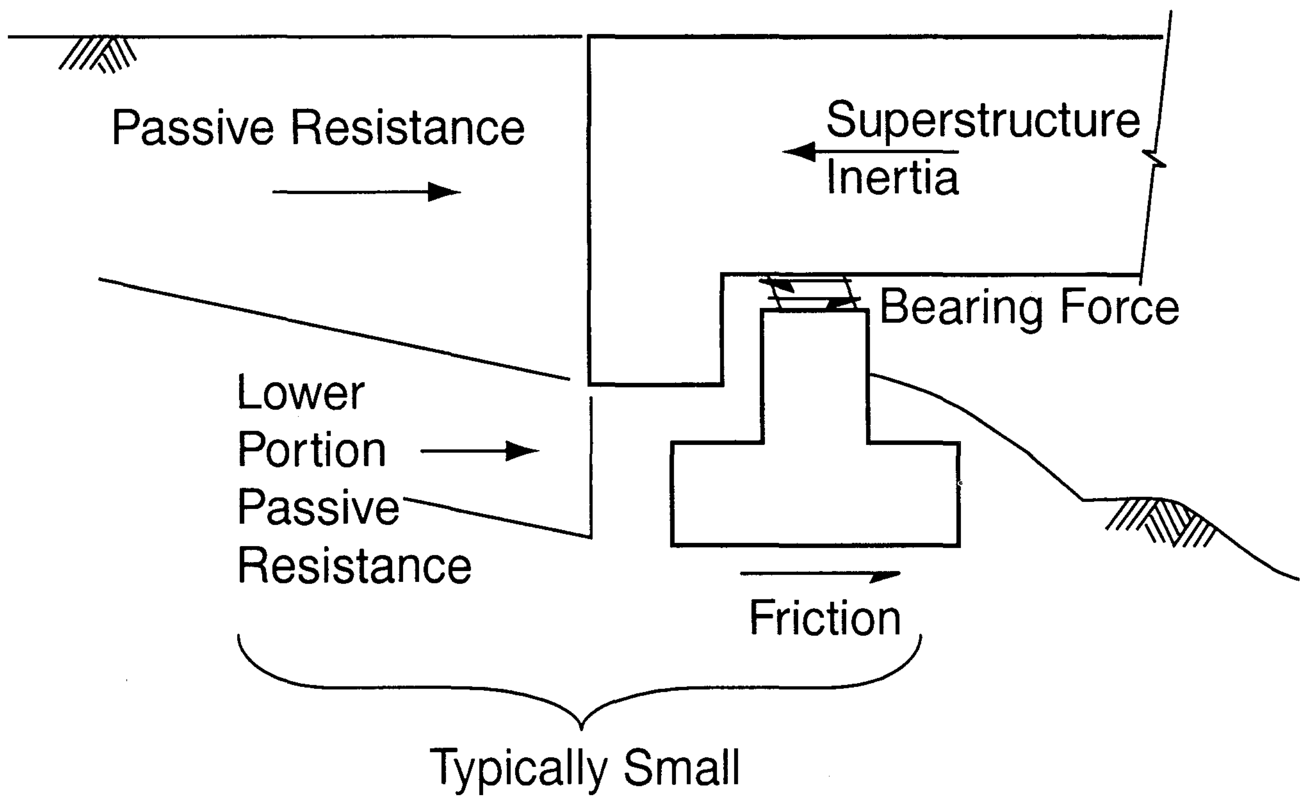
Superstructure Moves **Away** from Backfill



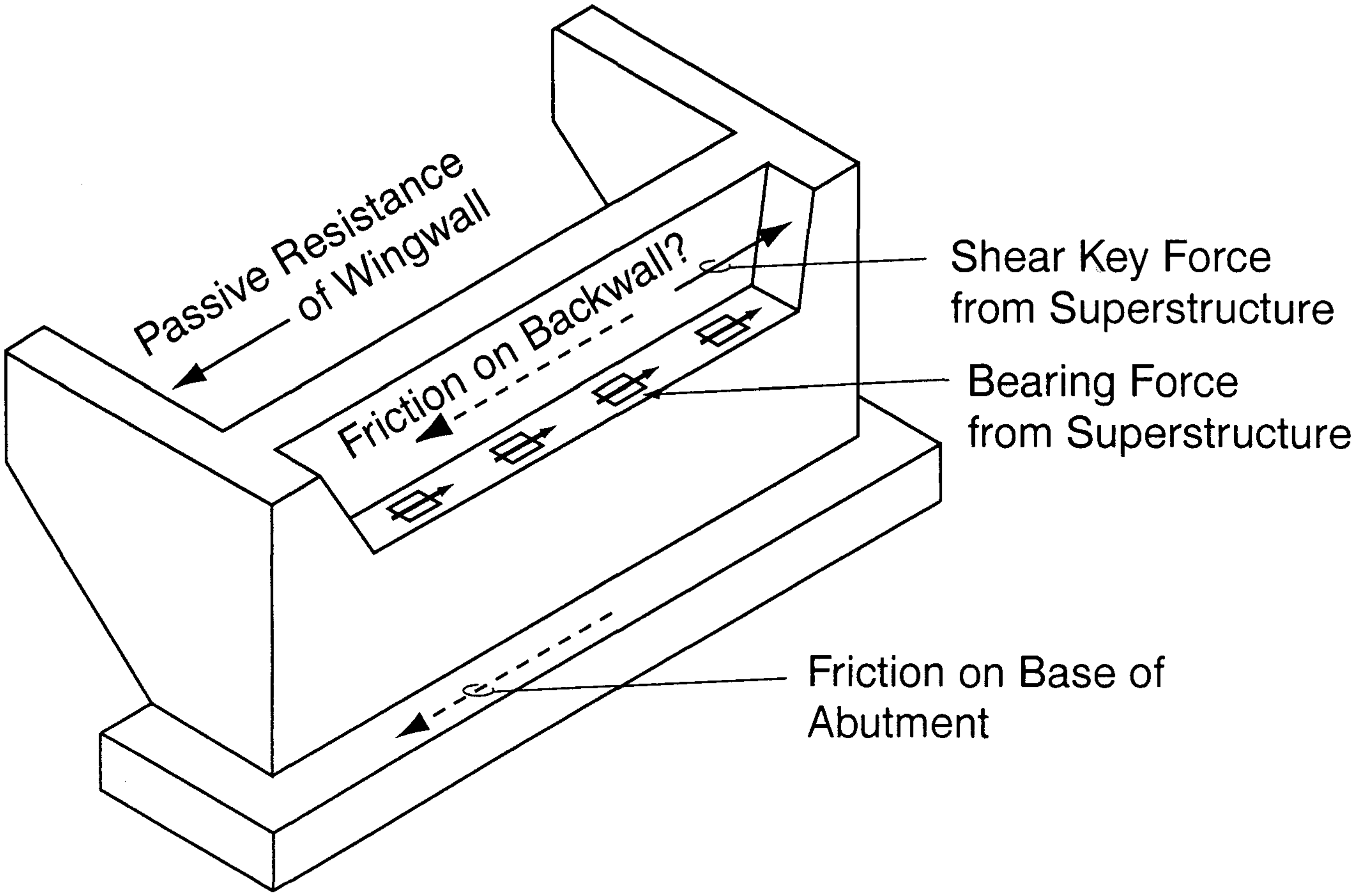
Effect of Piles Supporting Abutment



Integral Type / Longitudinal Behavior



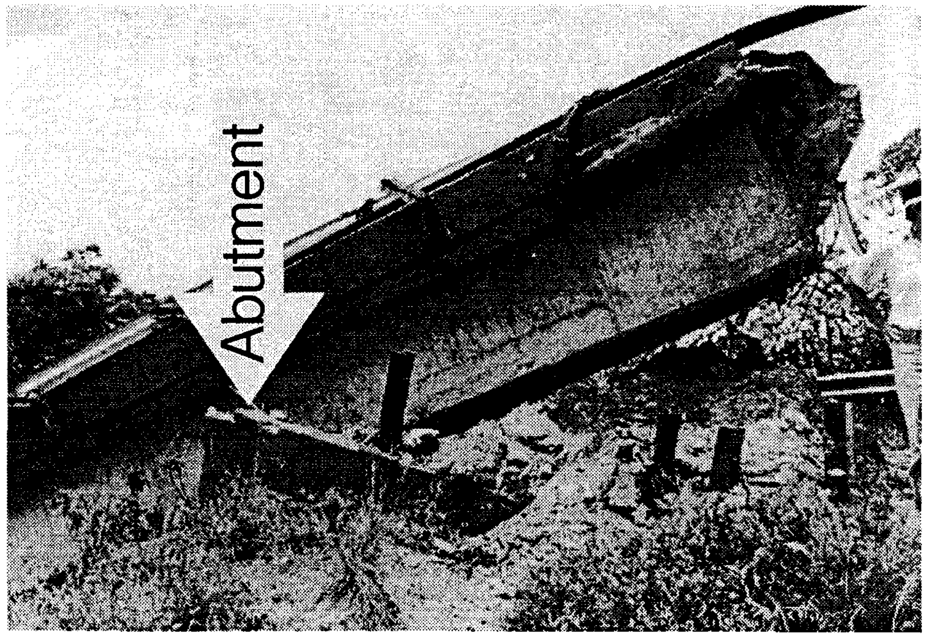
Transverse Behavior



Abutment Damage

Abutment Slumping and Rotation

Costa Rica, 1991



Passive Failure

Priestley, Seible, Calvi (1996)

Session 2
Abutments

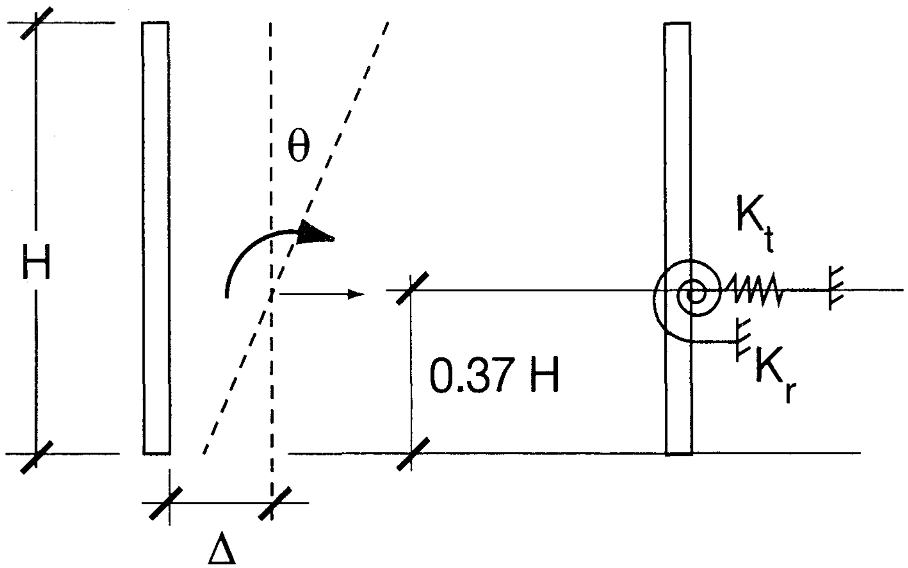
- Conceptual Behavior
- **Modeling Soil Flexibility**
- Nonlinear Effects
- Mononobe-Okabe Analysis
- Design Issues, Force Transfer, and Fuse Elements

Methods of Determining Stiffness

- Elasticity — FHWA / RD-86 / 101 (1986)
- Empirical — Caltrans

Focus on Elastic Stiffness First, Then Incorporate Nonlinear Behavior

FHWA Method



$K_t = 0.425\ E_sB$

$K_r = 0.072\ E_sBH^2$

E_s = Elastic Modulus of Backfill

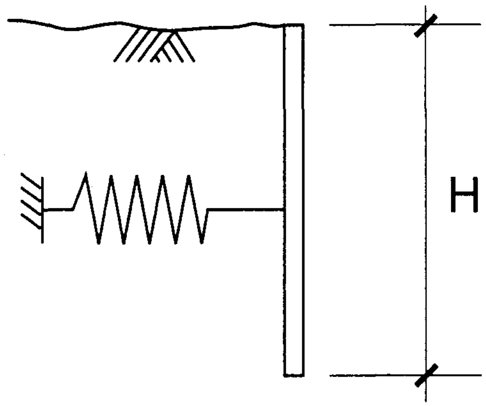
B = Width of Wall

H = Height of Wall

FHWA (1986)

Caltrans Method

- **Basic Stiffness**



$$K_{abut} = 200 \frac{\text{kip/in.}}{\text{ft of Width}}$$

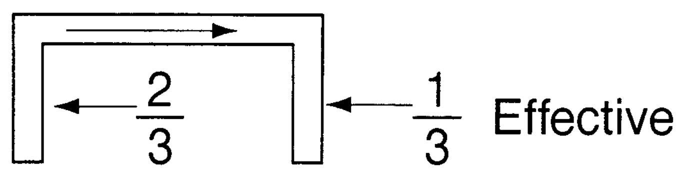
(8 ft High Wall)

- **Wall Height $\neq$ 8 ft**

  Linearly Prorate

- **Wingwalls**

  Assume $\frac{2}{3}$ Effective into Backfill, and $\frac{1}{3}$ Effective Away from Backfill



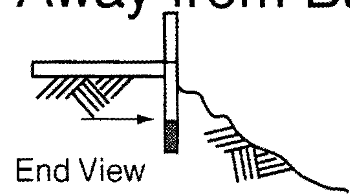
Caltrans Method (continued)

- Maximum Soil Capacity = 7.7 ksf (Passive)

Based on
- Properly Compacted and Drained Backfill
- Maximum Static = 5.0 Amplified by 1/0.65 for Dynamic Effects

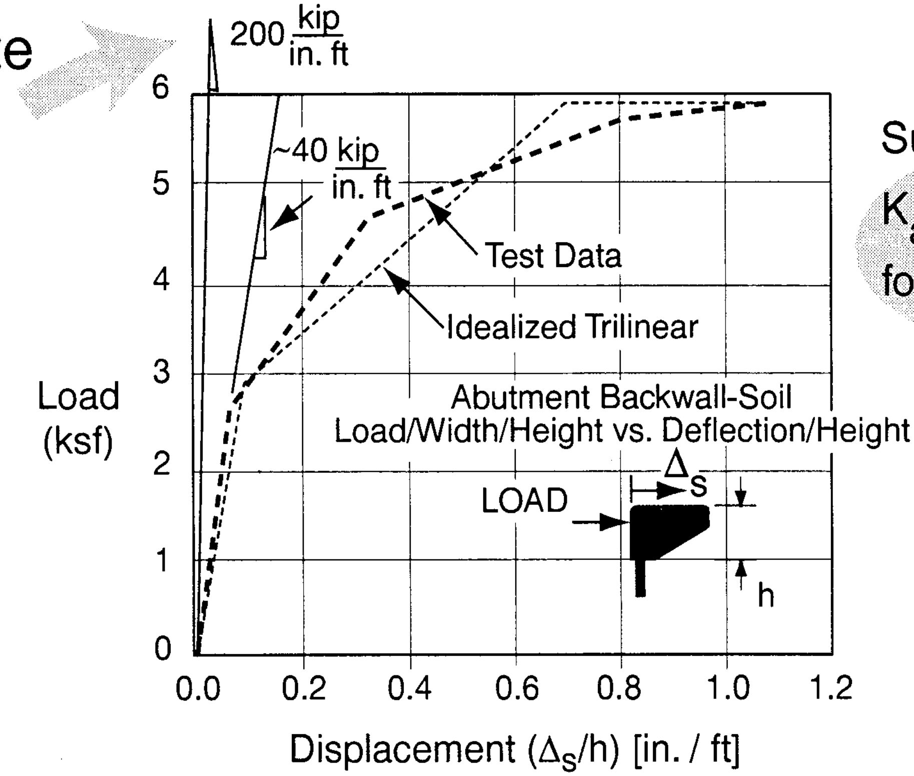
- Thoughts on Wingwalls
  - Effectiveness Acting Away from Backfill

$$\frac{1}{3} \longrightarrow 0\ ?$$



Test Data / Large Scale Abutment Tests

Note K



Suggest:

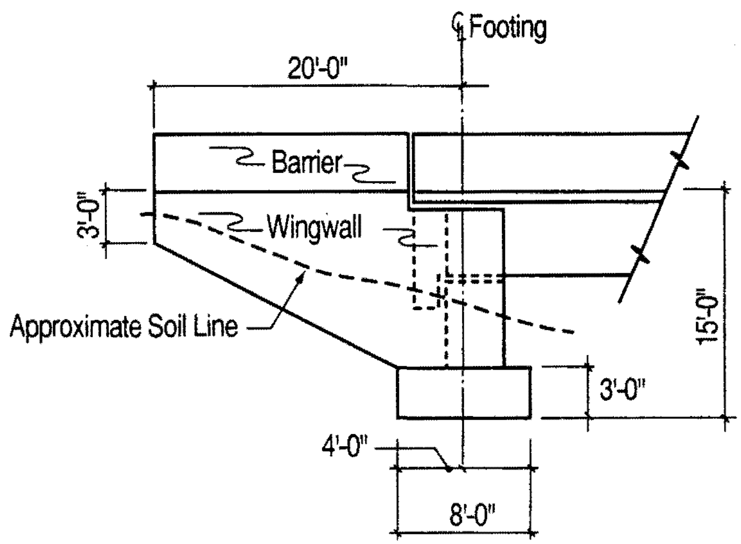
$K_{abut} \approx 40 - 50 \dfrac{kip}{in.\ ft}$

for an 8 ft High Wall

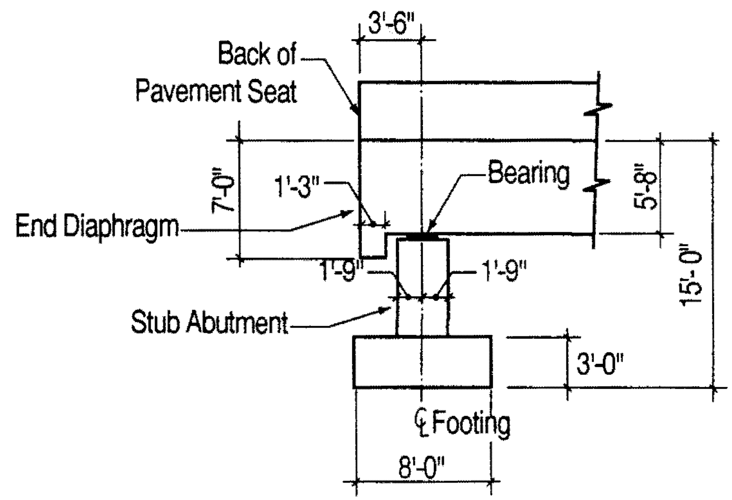
Priestly, Seible, Calvi, 1996

Example / Calculation of Abutment Stiffnesses

Consider Practice Problem No. 1 with an Integral Abutment



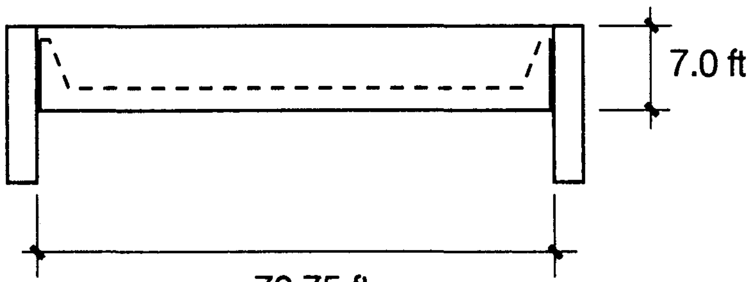
Elevation



Interior Section

Example / Abutment Stiffness (continued)

Assume the Following Geometry Between the Wingwalls



- Caltrans $K_{abut} = 40(73.75)\frac{7}{8}(12) = 30{,}975$ kip/ft

$$\frac{\text{kip/in.}}{\text{ft}}$$

Example / Effect of Abutment Stiffness On Seismic Forces

- Recall $K_{bent} = 3639$ kip/ft, $W = 4842$ kip } Session 1
- New Stiffness (Caltrans) $K_{total} = K_{bent} + K_{abut}$ — Consider One Abutment Acts at a Time
  $K_{total} = 34{,}614$ kip/ft
- New Values $V = 1816$ kip — { 191 kip to Bent (vs. 886 kip Before!); 1625 kip to Soil }
  $\Delta = 0.63$ in.

Example / Check of Abutment Behavior

- Determine Backfill Pressure

$$p = \frac{1625}{7(73.75)} = 3.15 \text{ ksf} < 7.7 \text{ ksf Capacity}$$

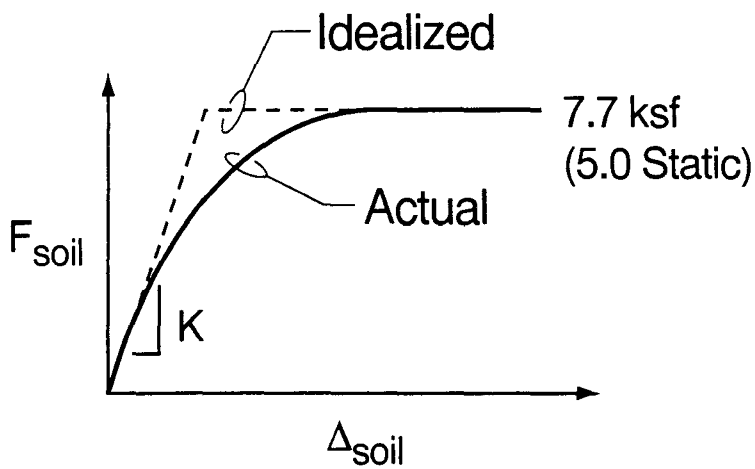
$$\therefore \text{ OK!}$$

Soil Can Withstand Forces
in Longitudinal Direction

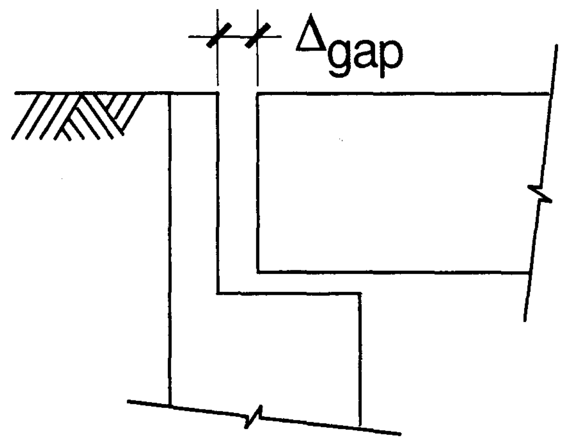
Session 2
Abutments

- Conceptual Behavior
- Modeling Soil Flexibility
- **Nonlinear Effects**
- Mononobe-Okabe Analysis
- Design Issues, Force Transfer, and Fuse Elements

Sources of Nonlinearity

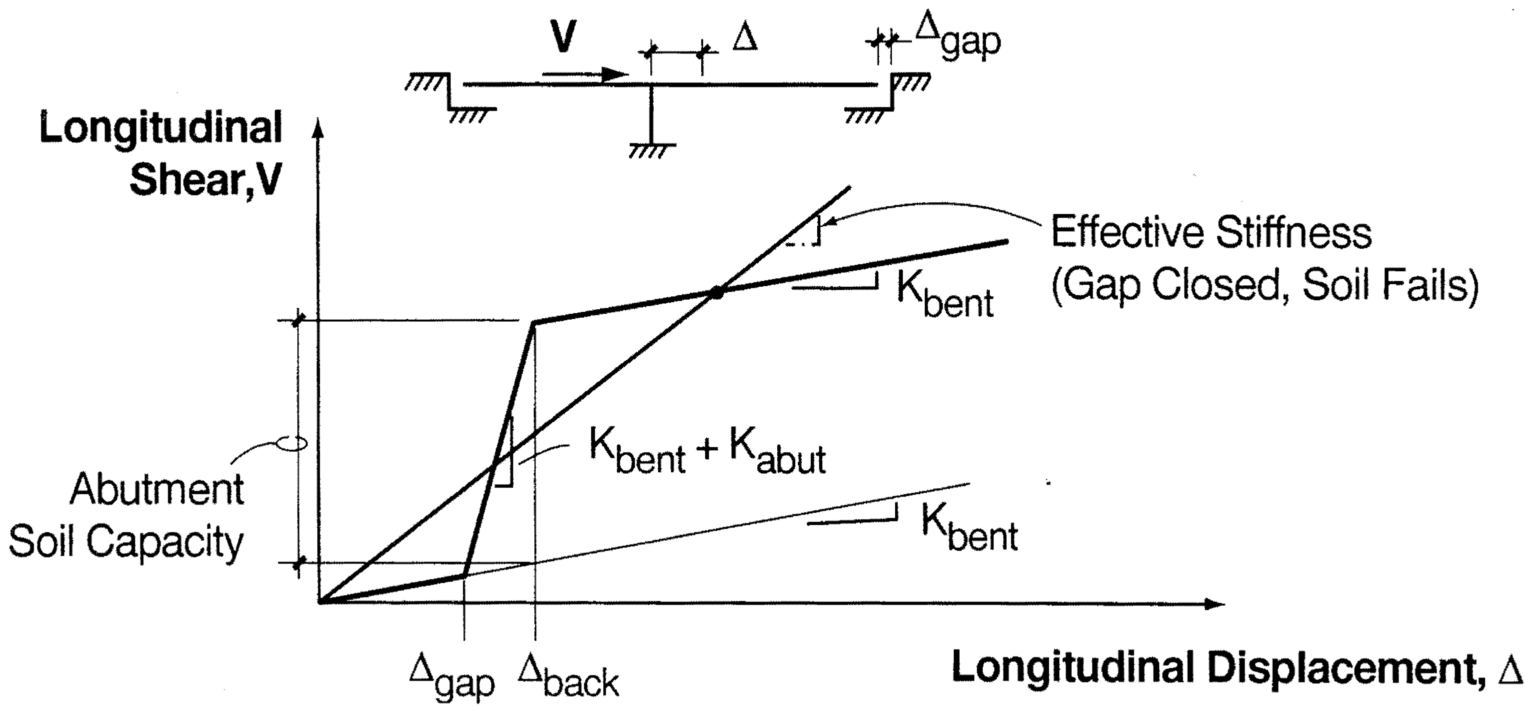


Soil Behavior



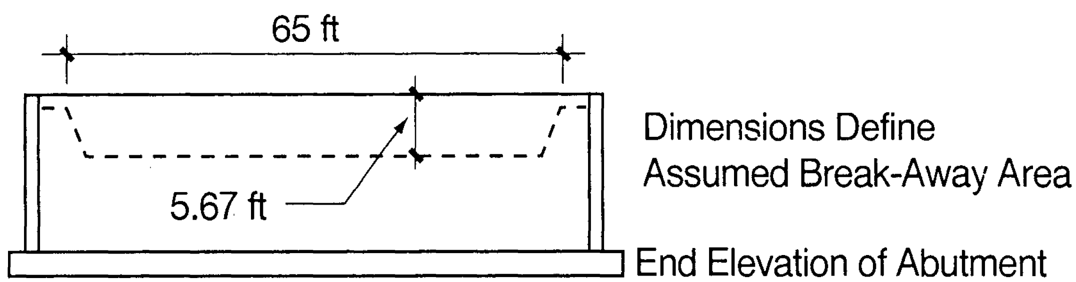
Movement Joints

Overall Structure Stiffness



Example / Abutment Nonlinearities (1 of 7)

- Use Seat Abutment Detail Given with Practice Problem No. 1
- Leave Columns at 3 ft Diameter
- Assign A = 0.40g (In Order to Be Well into Nonlinear Range)
- Assume Backwall Breaks Away Around Perimeter of Box Girder
- Recall $K_{bent} = 3639 \frac{\text{kip}}{\text{ft}}$, $\Delta_{gap} = 6$ in., S = 1.2, and W = 4842 kip



Example / Abutment Nonlinearities (2 of 7)

- Longitudinal Stiffness of Abutment (Caltrans)

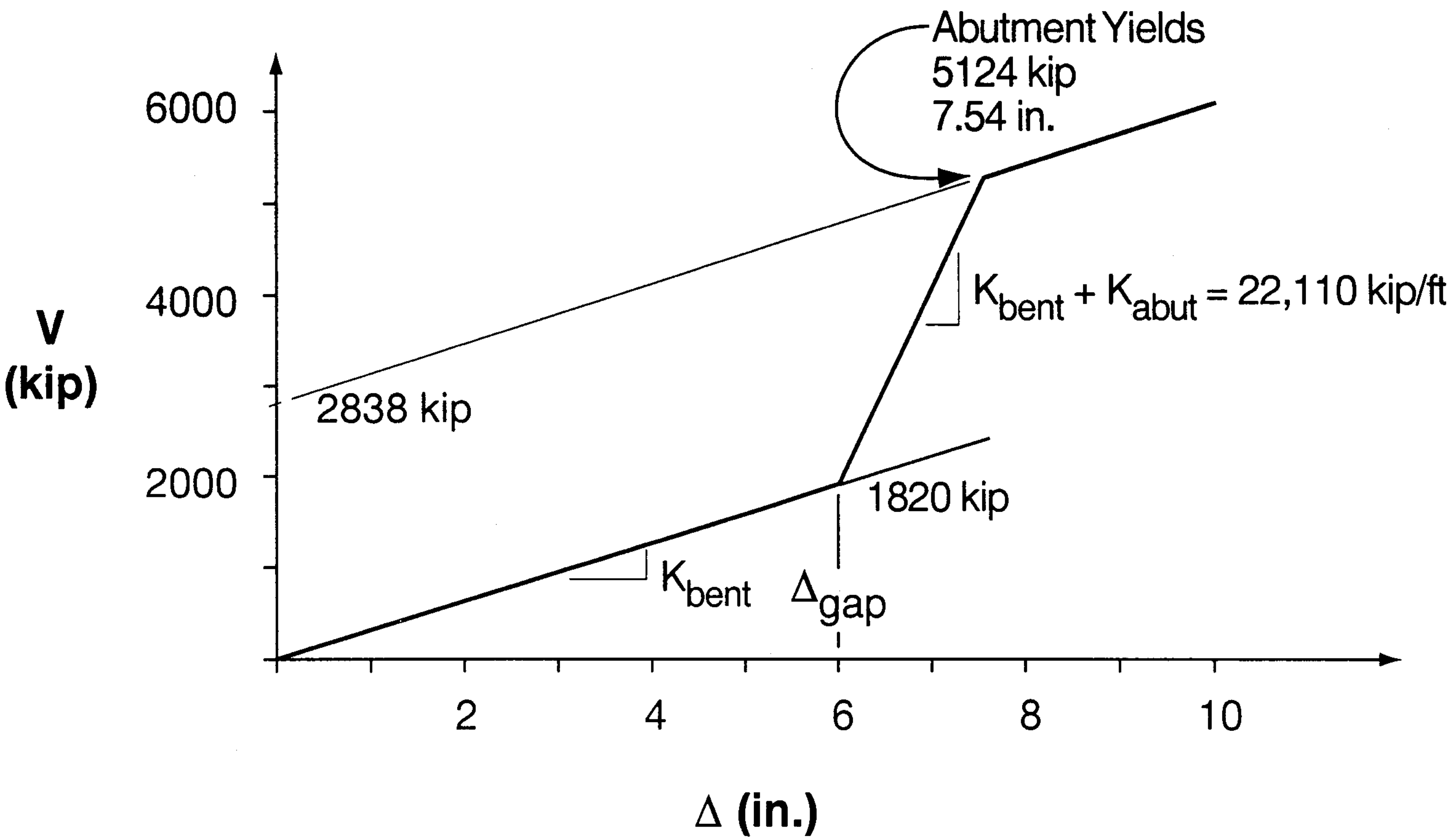
  $$K_{abut} = 40\ (65)\ \frac{5.67}{8}\ (12) = 22{,}110 \text{ kip/ft}$$

- Abutment Backfill Capacity (Caltrans)

  $$P_{max} = 7.7(65)5.67 = 2838 \text{ kip}$$

- Construct V vs. Δ Curve for Structure (Longitudinal)

Example / Abutment Nonlinearities (3 of 7)



Example / Abutment Nonlinearities (4 of 7)

- Check Δ with Only Bent

  $K = 3639$ kip/ft $\longrightarrow$ $T = 2\pi\sqrt{\dfrac{4842}{32.2(3639)}} = 1.28$ sec

- $C_S = \dfrac{1.2\,(0.4)1.2}{1.28^{2/3}} = 0.49 < 1.00 \longrightarrow V = 0.49(4842) = 2373$ kip

- $\Delta = \dfrac{2373}{3639}(12) = 7.8 \text{ in.} > 6 \text{ in.}$ $\therefore$ Into Nonlinear Range

- Iterative Approach — Guess K, Determine V and Δ, Revise

- Direct Approach — Plot Spectral V vs. Δ

Example / Abutment Nonlinearities (5 of 7)

Direct Spectral Approach

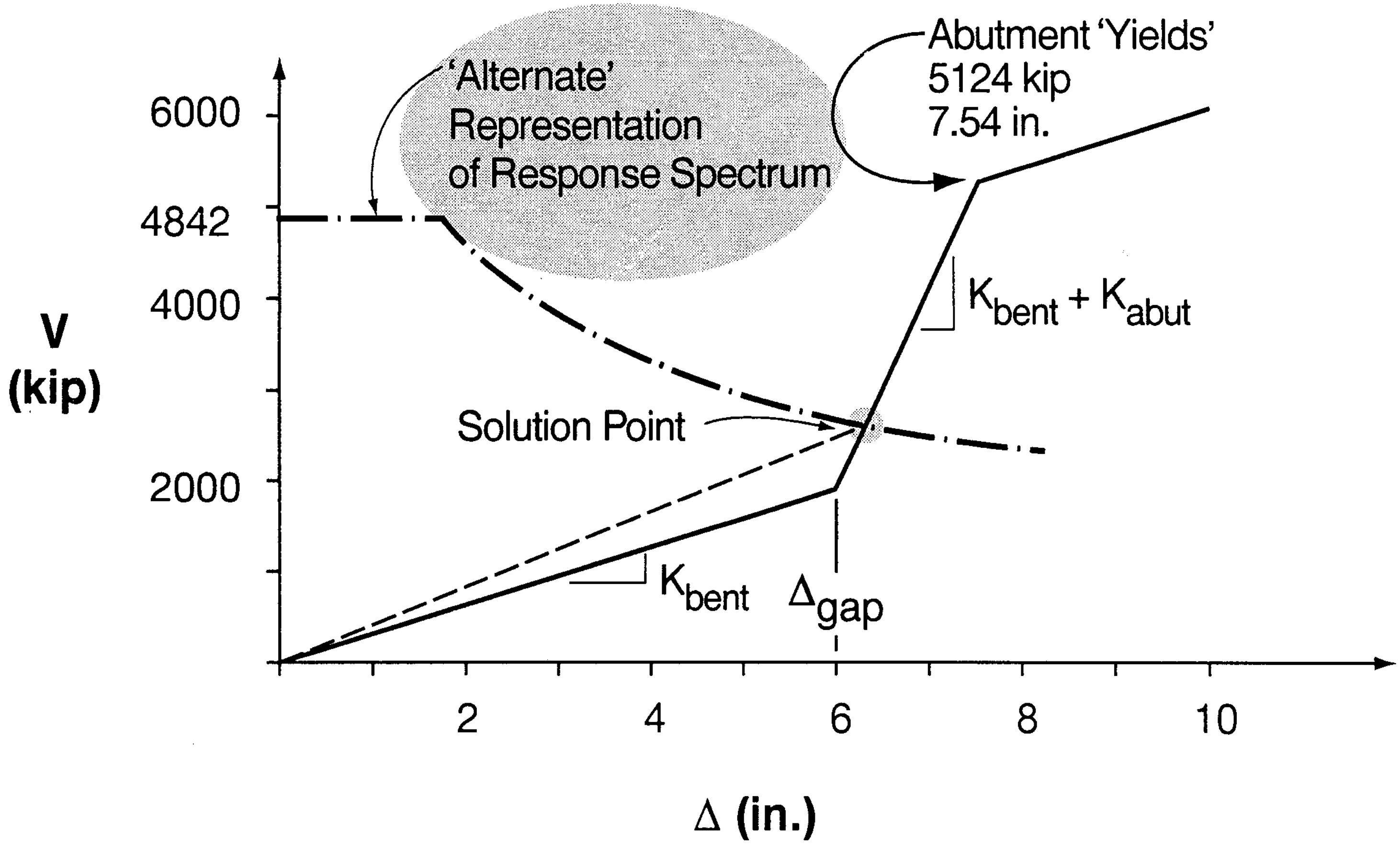
- $V = f(C_S)$ $C_S = f(T)$ $T = f(W/K)$ $K = f(V/\Delta)$ $\therefore \mathbf{V = f(\Delta)}$
- For a SDOF System with Full Mass Participation ($V = C_S W$)

$$V = \frac{(1.2AS)^{3/2} W\, g^{1/2}}{2\pi} \frac{1}{\Delta^{1/2}} \leq 2.5\ AW$$

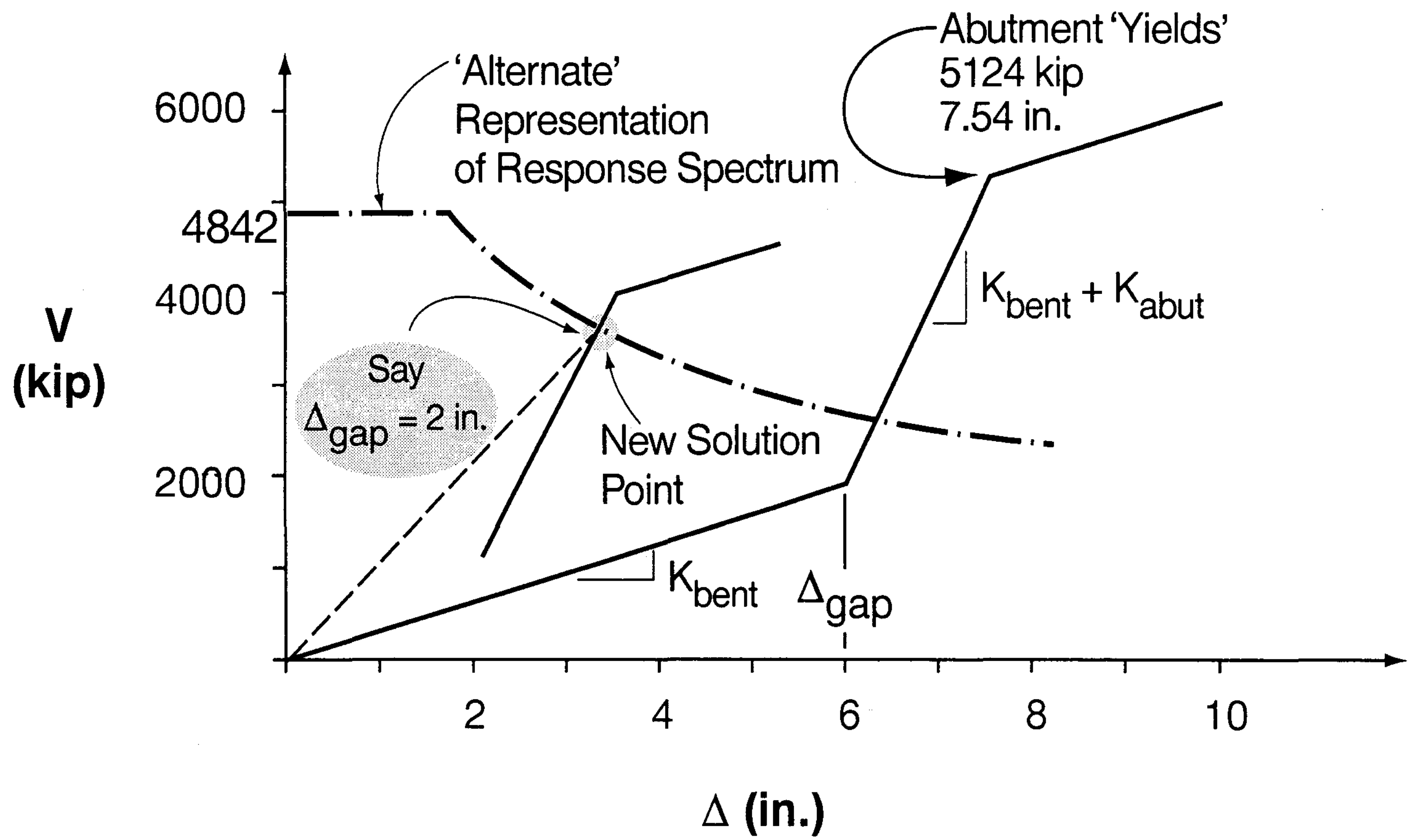
- For This Example

$$V = 1912 \frac{1}{\Delta^{1/2}} \leq 4842 \text{ kip} \quad (\Delta \text{ in ft})$$

Example / Abutment Nonlinearities (6 of 7)



Example / Abutment Nonlinearities (7 of 7)

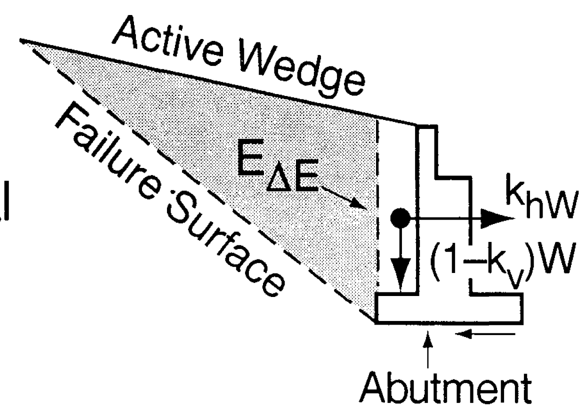


Session 2
Abutments

- Conceptual Behavior
- Modeling Soil Flexibility
- Nonlinear Effects
- **Mononobe-Okabe Analysis**
- Design Issues, Force Transfer, and Fuse Elements

Pseudostatic Approach / Yielding Abutments

- Applies to Seat-Type (Freestanding) Abutments that Are Not Restrained by Superstructure
- Cohesionless Backfill with Friction Angle ϕ
- Unsaturated / No Liquefaction
- Coulomb Sliding Wedge + Vertical and Horizontal Inertia Effects



Calculation of Active Seismic Loading on Wall

$$E_{AE} = \frac{1}{2}\,\gamma H^2(1-k_v)K_{AE}$$

- Inertial Effect Increases Forces

γ = Soil Unit Weight

H = Wall Height

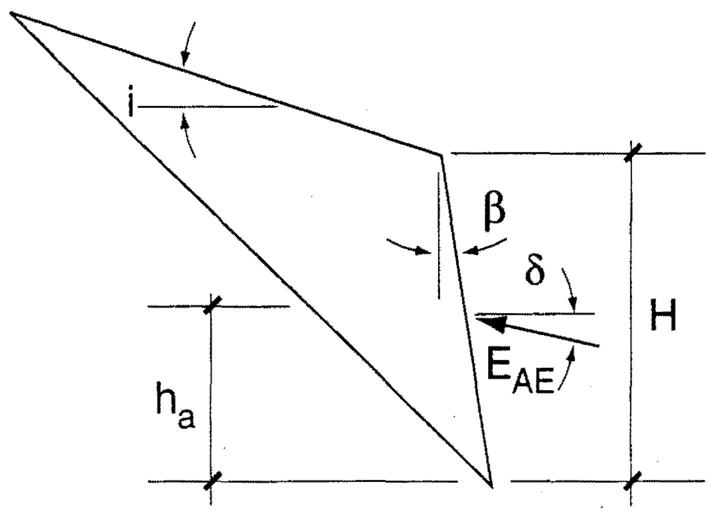
k_v = Vertical Acceleration Coefficient

k_h = Horizontal Acceleration Coefficient

Typically $\left.\begin{array}{l} k_v = 0 \\ k_h = 0.5A \end{array}\right\}$ Division I-A 6.4.3(A) and 7.4.3(A)

Active Seismic Loading (continued)

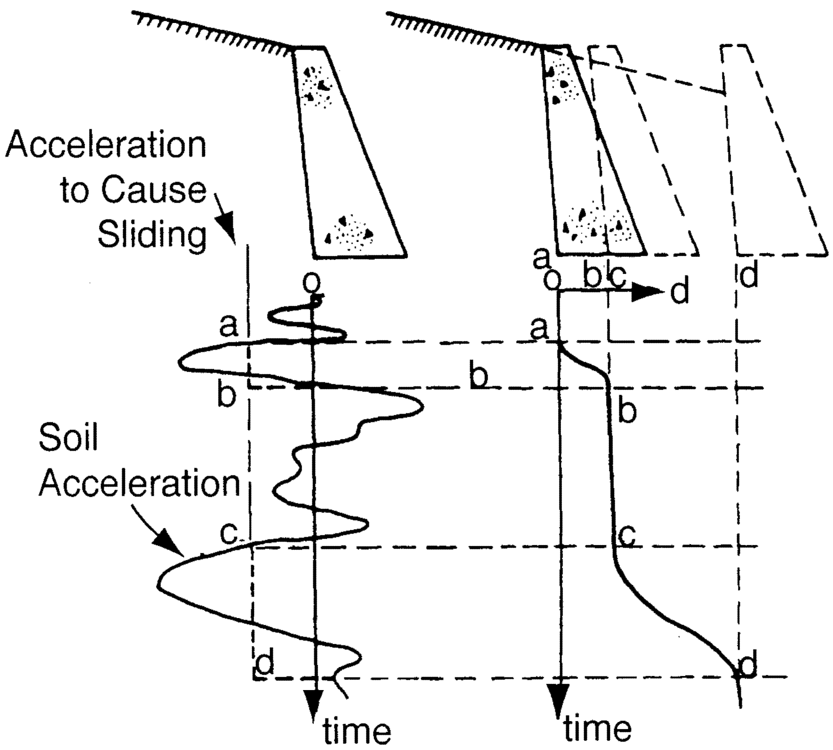
$$K_{AE} = \frac{\cos^2(\phi - \theta - \beta)}{\cos\theta \cos^2\beta \cos(\delta + \theta + \beta)\left[1 + \sqrt{\dfrac{\sin(\theta + \delta)\sin(\phi - \theta - i)}{\cos(\delta + \beta + \theta)\cos(i - \beta)}}\right]^2}$$



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

$$\delta = \frac{\phi}{2} \text{ (Typical Approximation)}$$

Allowing Some Wall Movement



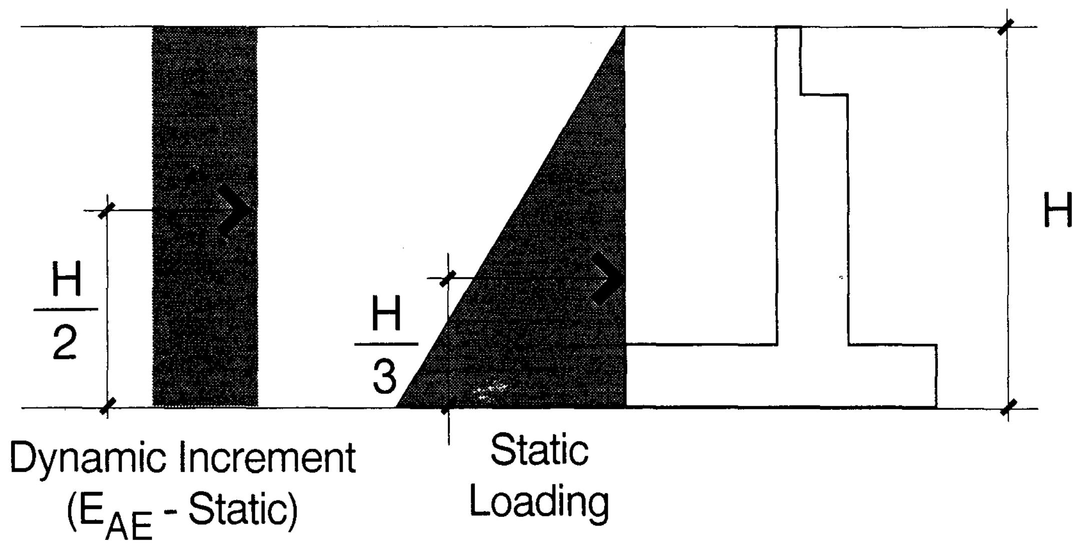
Acceleration **Displacement**

- By Allowing Some Movement, $k_h = 0.5$ A (Instead of A)
- Expect Displacements to 10 • A (in.)
- Also Basis of 7.7 ksf vs. 5.0 ksf Used by Caltrans

AASHTO (1994), Division I-A, Commentary

Distributing the Force

- M-O Expression Includes the Static Active Load
- Obtain Static Force by Using k_h (or θ) = 0



Other Conditions

- Abutments Restrained by Soil Anchors or Battered Piles, Use $k_h = 1.5A$
- Abutments Moving into Soil, — Could Use M-O Passive, But No Experimental Verification

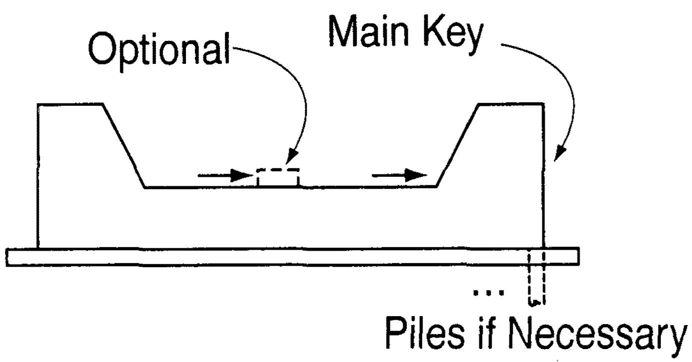
Using the Concepts

Abutment Type / Condition		Method	...	Product
• Seat / Gap Open		M-O Active	...	Loading
• Seat / Gap Closed		Caltrans	...	Stiffness / Capacity
	or			
		FHWA	...	Stiffness
• Integral		Caltrans	...	Stiffness / Capacity
	or			
		FHWA	...	Stiffness

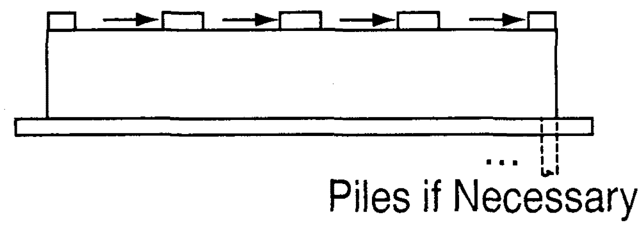
Session 2
Abutments

- Conceptual Behavior
- Modeling Soil Flexibility
- Nonlinear Effects
- Mononobe-Okabe Analysis
- **Design Issues, Force Transfer, and Fuse Elements**

Transverse Loading of Abutments / Shear Keys



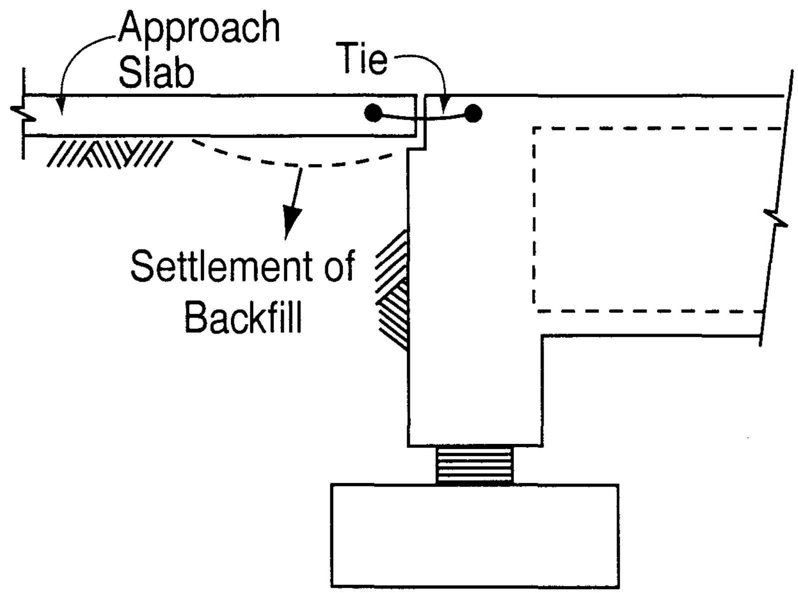
Box Girder Arrangement



Precast or Steel Girder Arrangement

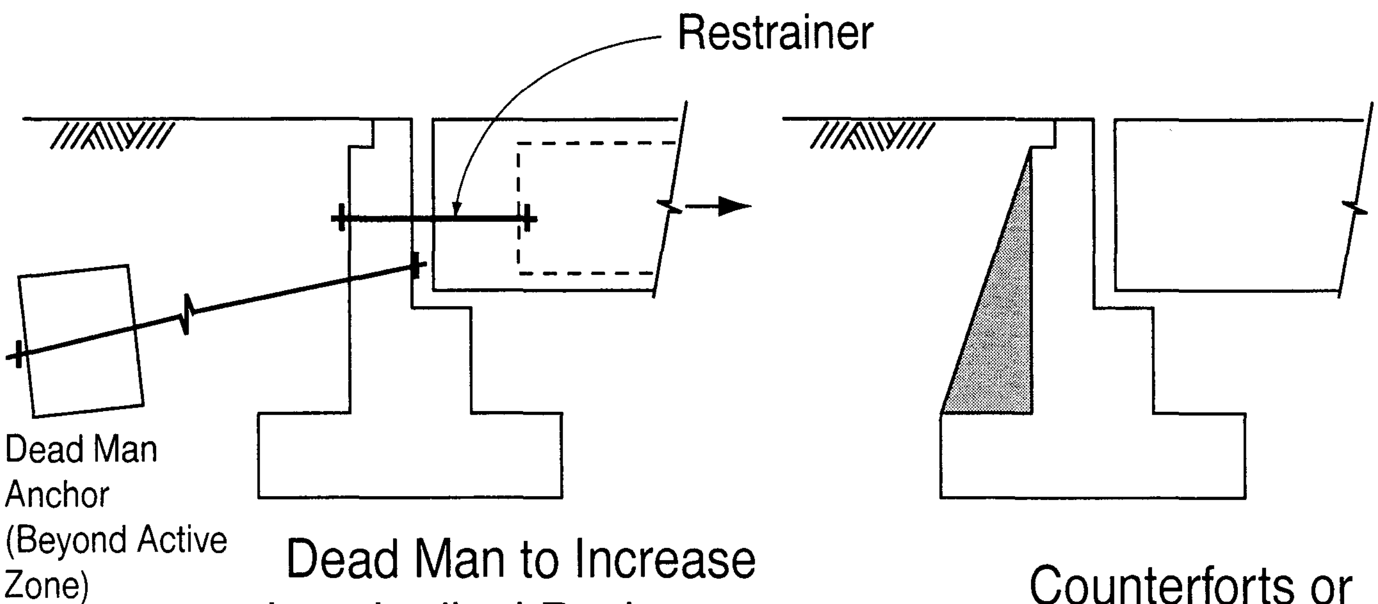
- Interior Keys for Box Girders Difficult to Inspect and Repair
- Multiple Keys May Not Load Evenly (Be Conservative / Ductile)
- Consider 'Fusing' Keys to Fail Before Damaging Piles

Approach Slabs



- If Settlement Occurs, Approach Slab Provides Access to Bridge (Required for SPC **D**, Emergency Response)
- Tie to Superstructure to Prevent Unseating

Enhancements for Force Transfer



Dead Man to Increase
Longitudinal Resistance

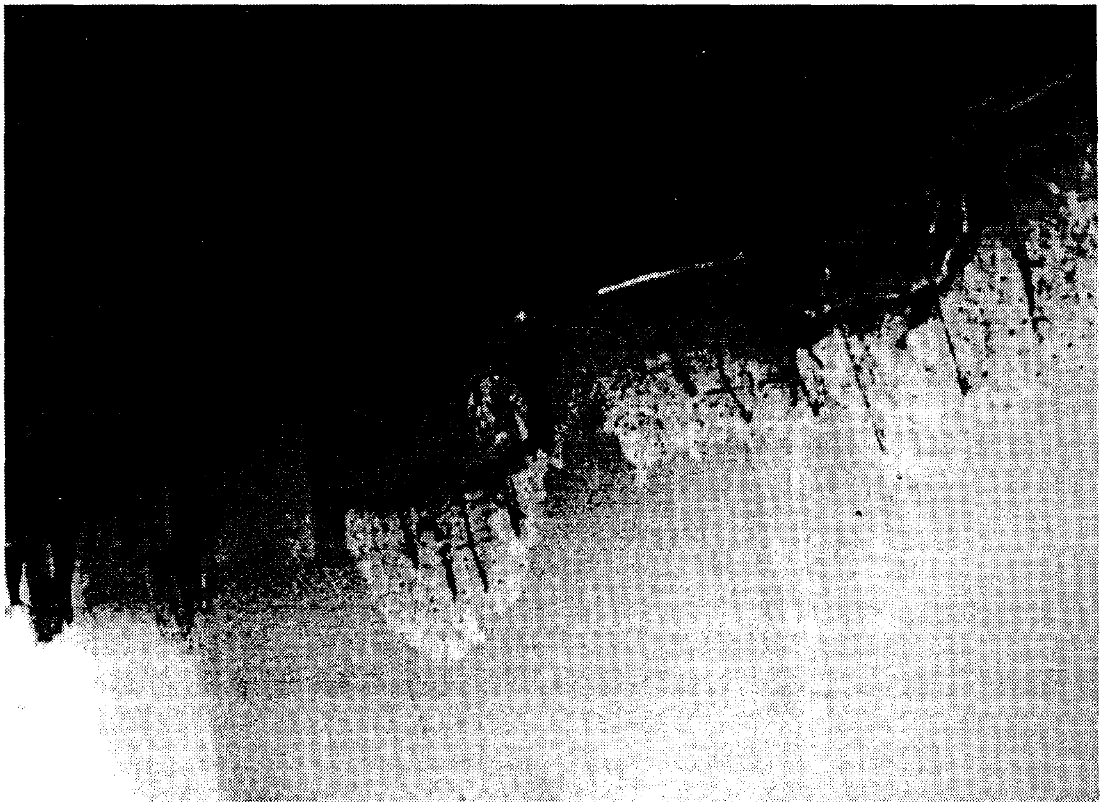
Counterforts or
Shear Lugs Increase
Transverse Resistance

External Shear Key Damage

EERI (1995)

Northridge, 1994

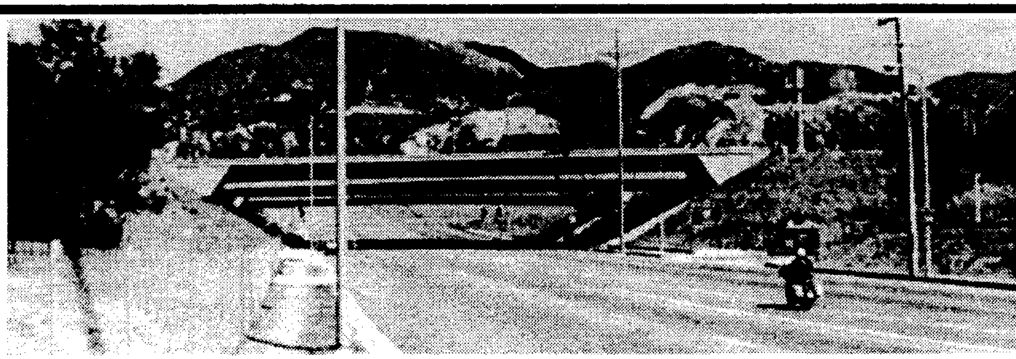
Internal Shear Key Damage



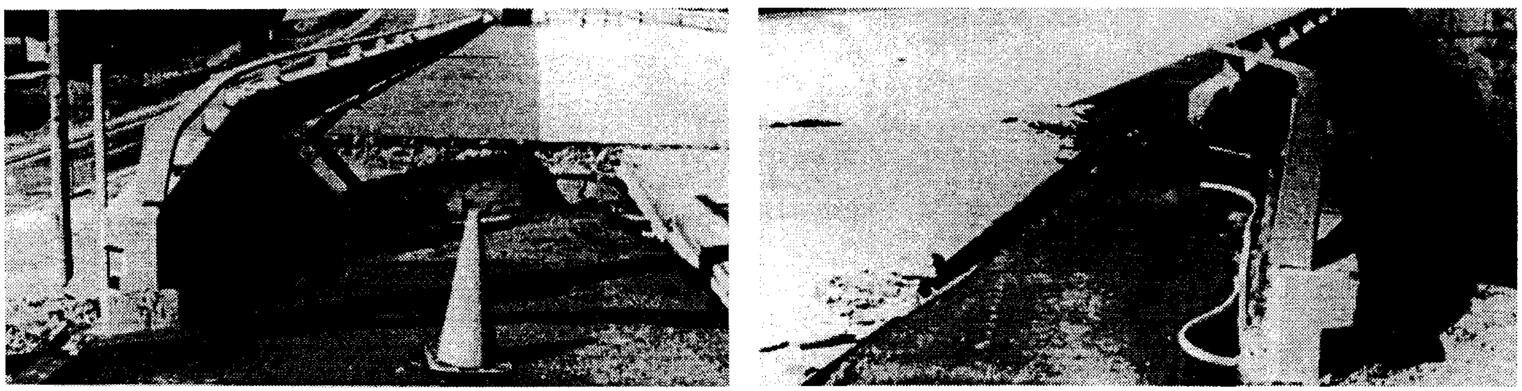
Northridge, 1994

EERI (1995)

Transverse Response and Backfill Settlement Issues

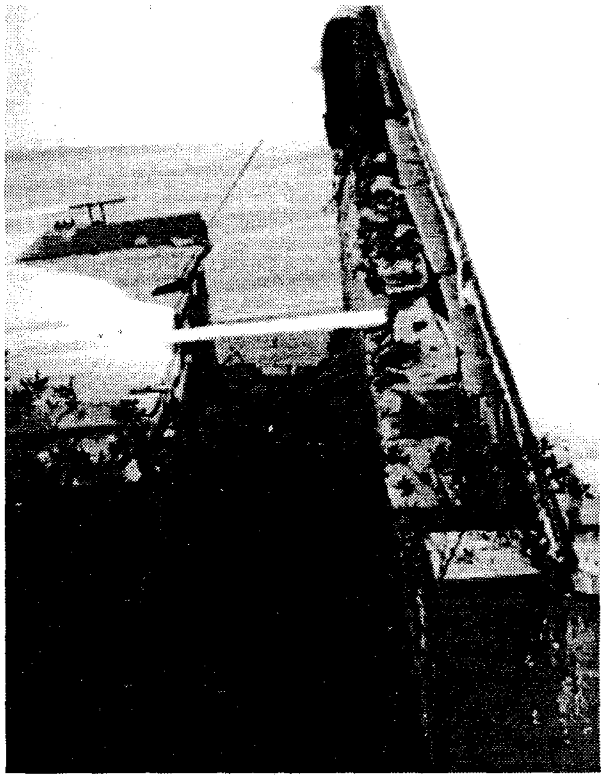


San Fernando, 1971



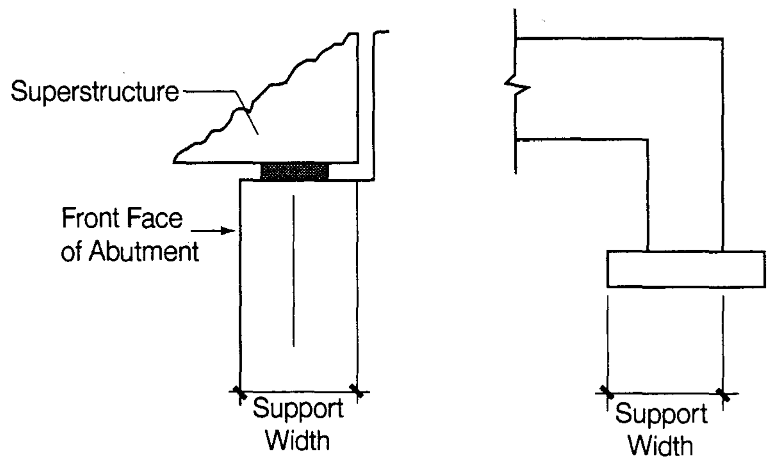
Caltech (1971)

Most Important of All – Seat Width



EERI (1995)

Northridge, 1994



Seat Abutment **Integral Abutment**

Session 3
Steel Plate Girder Bridge Examples

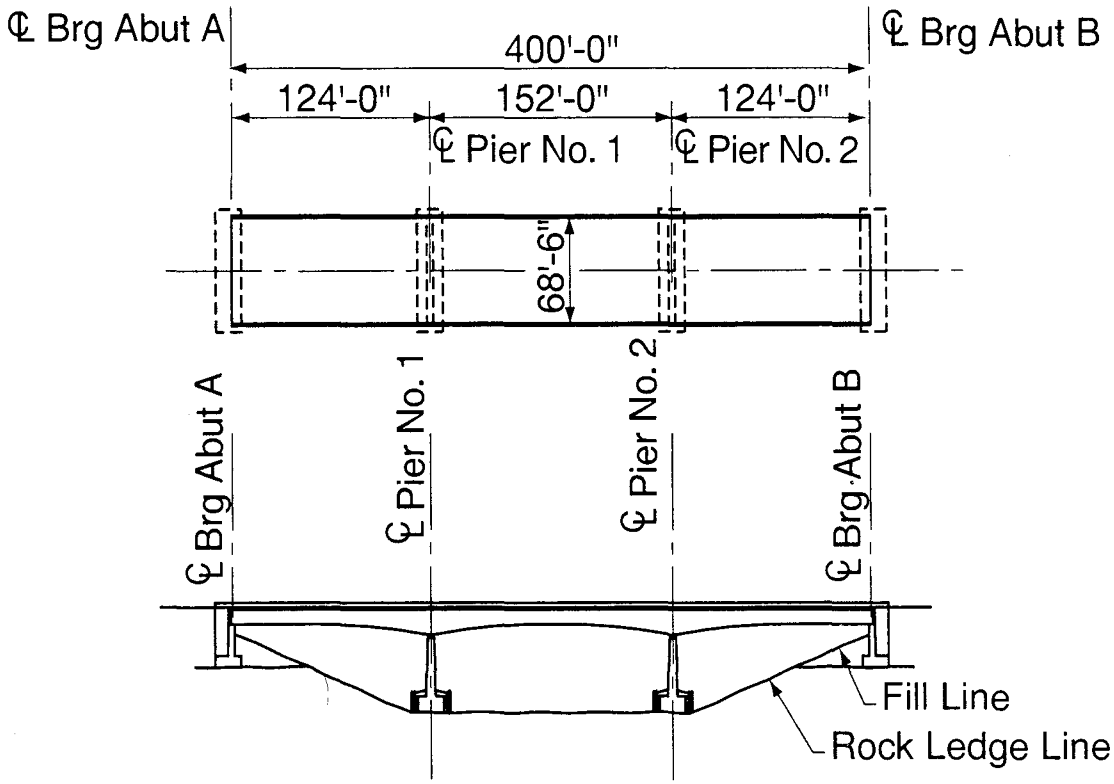
Session 3

- Practice Problem No. 2
- Conceptual Design Considerations
- Steel Superstructure Issues

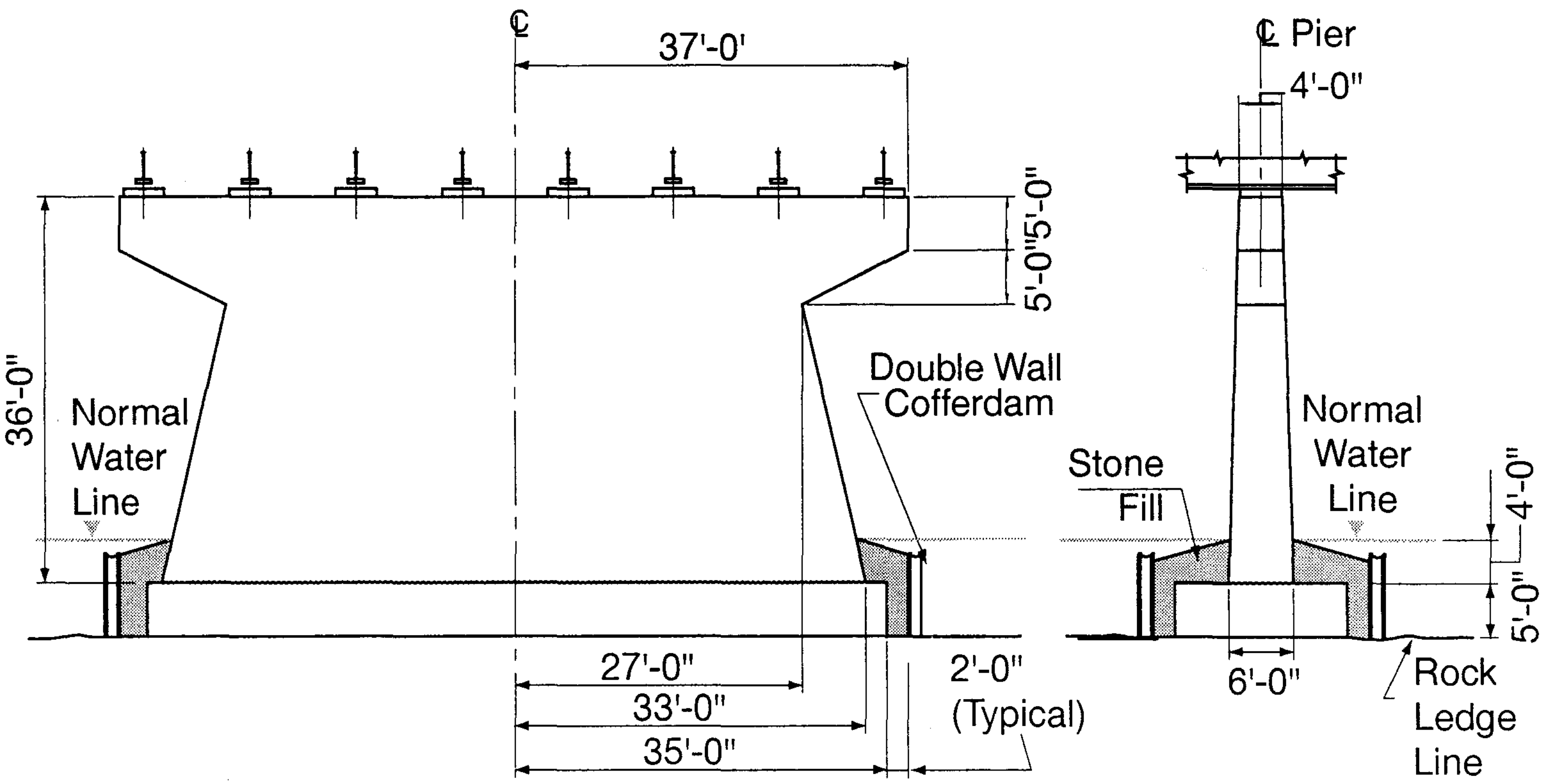
Session 4

- Skew Structure Issues
- Elastomeric Bearing Modeling

Steel Plate Girder Bridge / Layout

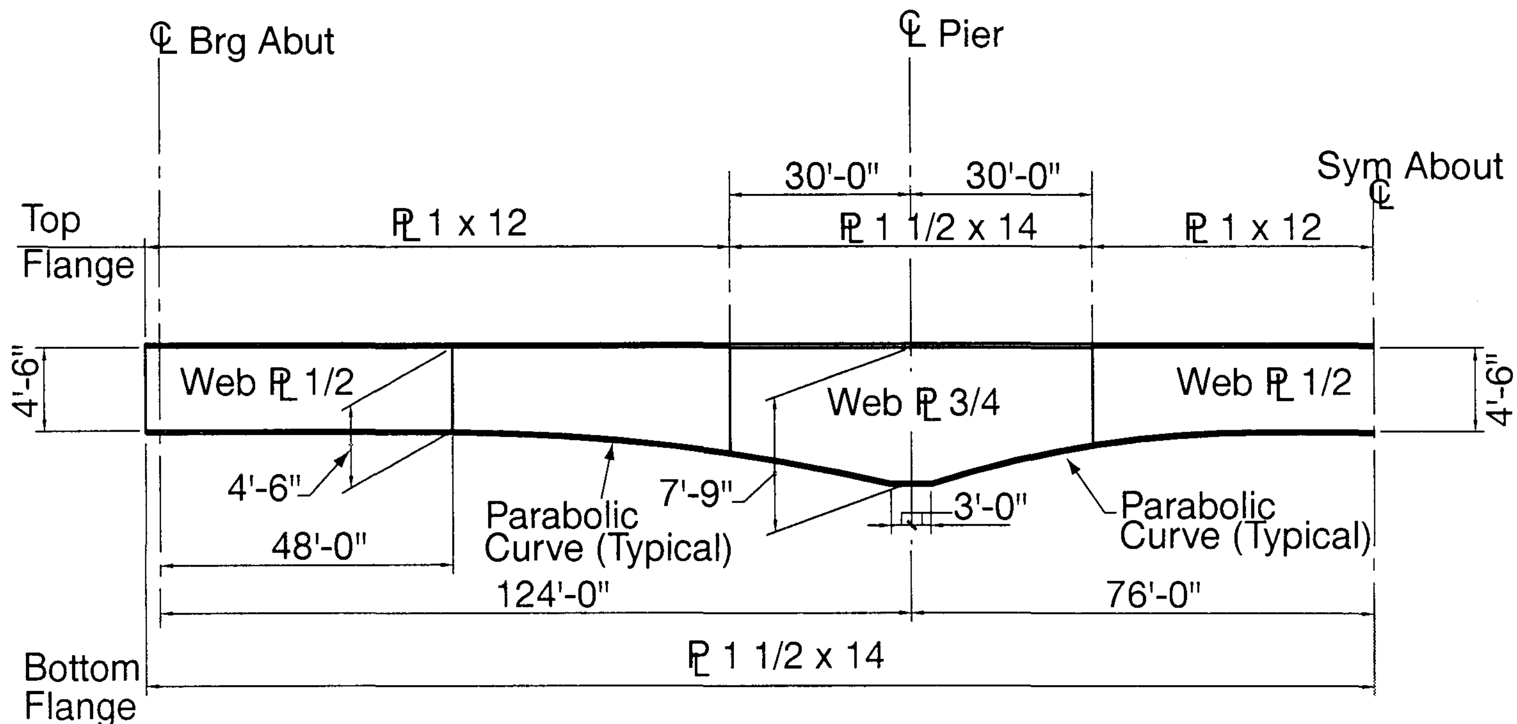


Steel Plate Girder Bridge / Wall Pier



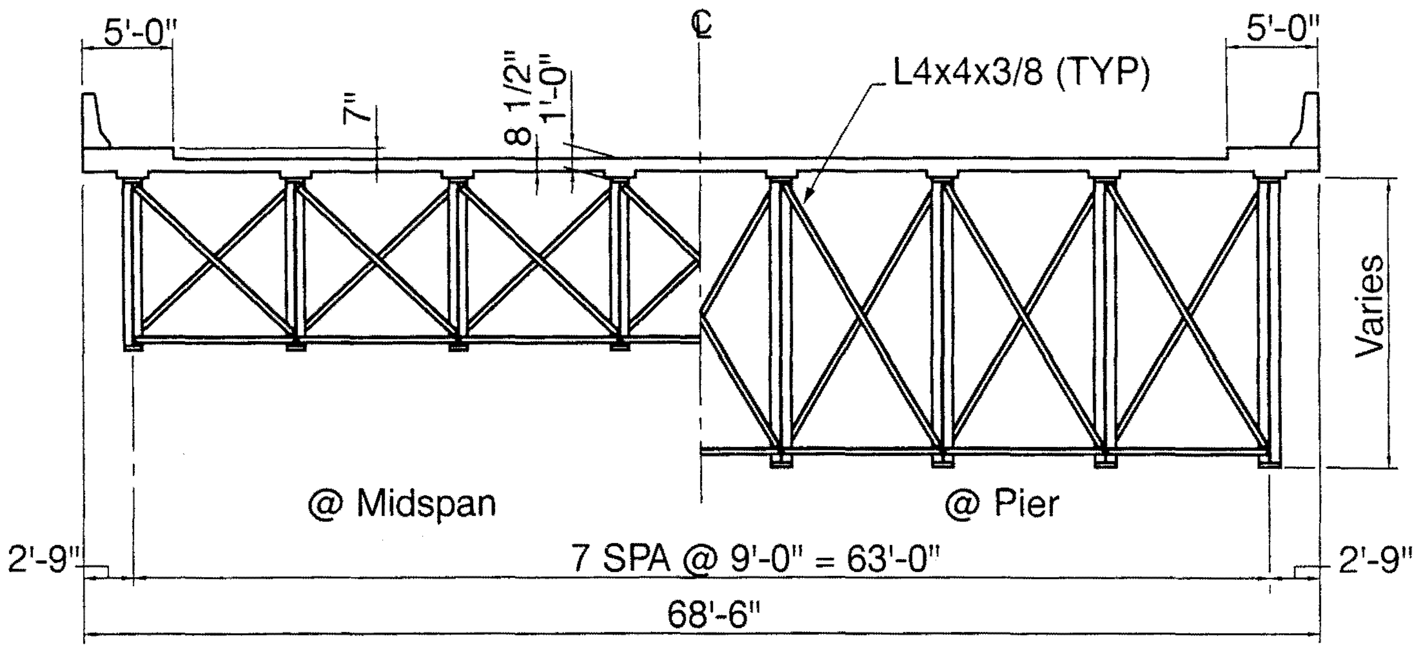
Pier Elevation **End Elevation**

Steel Plate Girder Bridge / Girder Elevation

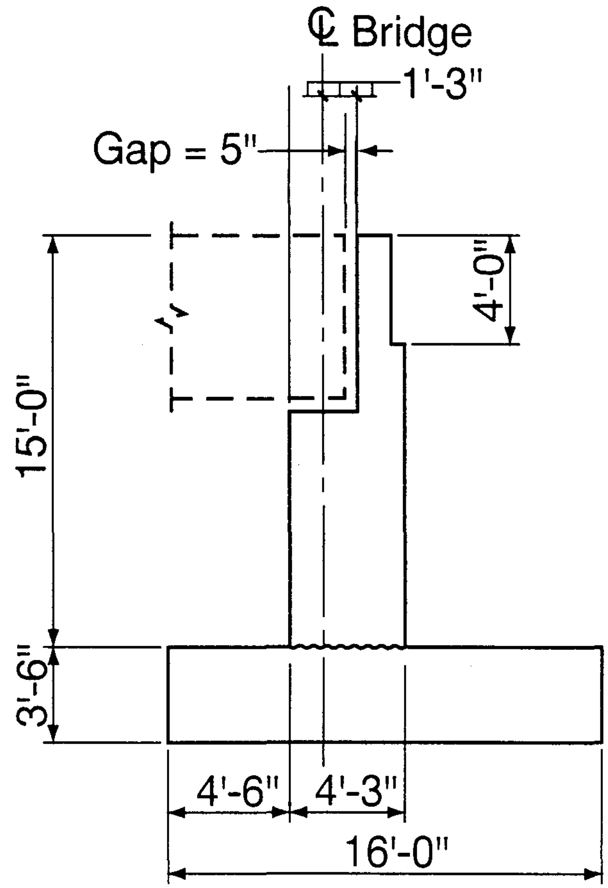


Girder Elevation

Steel Plate Girder Bridge / Superstructure Section



Steel Plate Girder Bridge / Abutment Section



Session 3
Required / Practice Problem No. 2

- Calculate Longitudinal Period
- Calculate Elastic Longitudinal Shear, Moment, and Displacement of Pier No. 1
- Design Pier No. 1 Reinforcement
- Size Footing
- Consider Alternatives

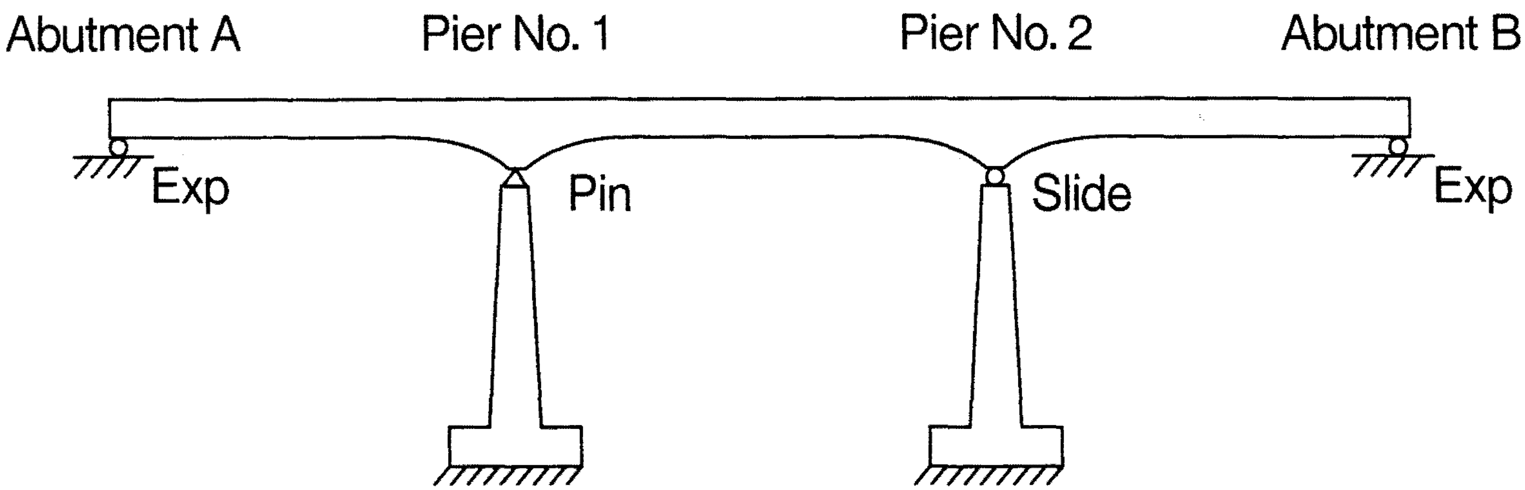
Basic Data for Bridge

- Acceleration Coefficient, A = 0.15g
- Seismic Performance Category, SPC = B
- Soil — Rock

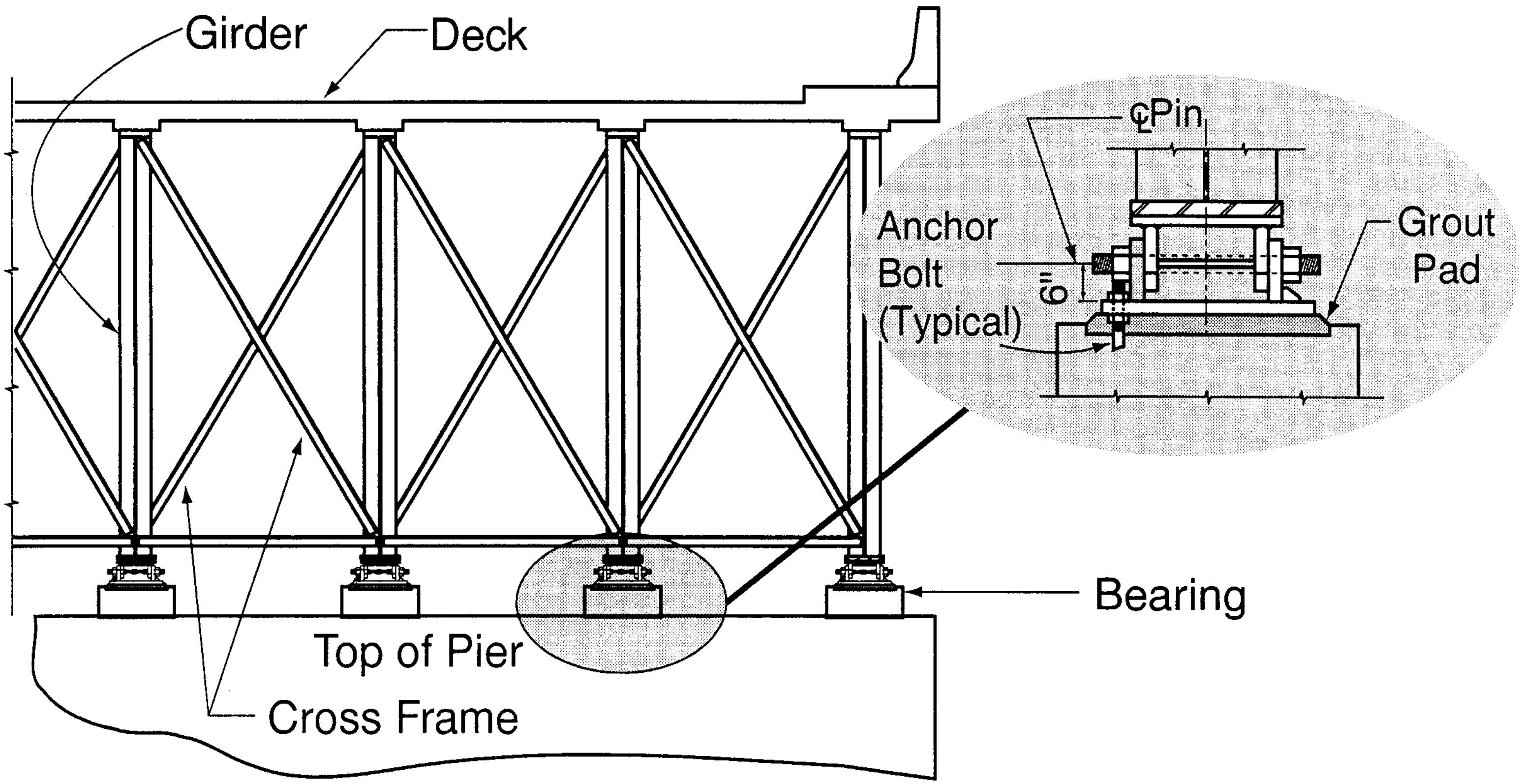
  $S = 1.0$

  $f_{ult} = 50$ ksf Ultimate Bearing Capacity

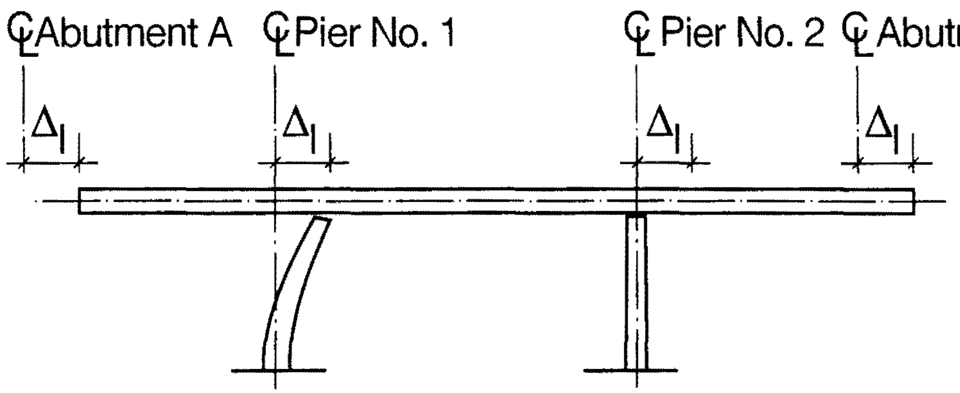
Bearing Conditions – Longitudinal



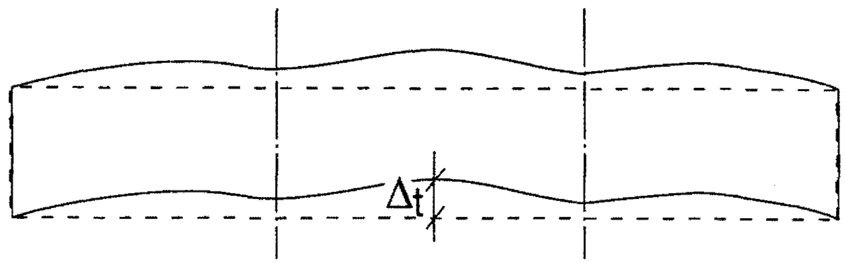
Bearing Conditions – Transverse



Expected Lateral Seismic Behavior

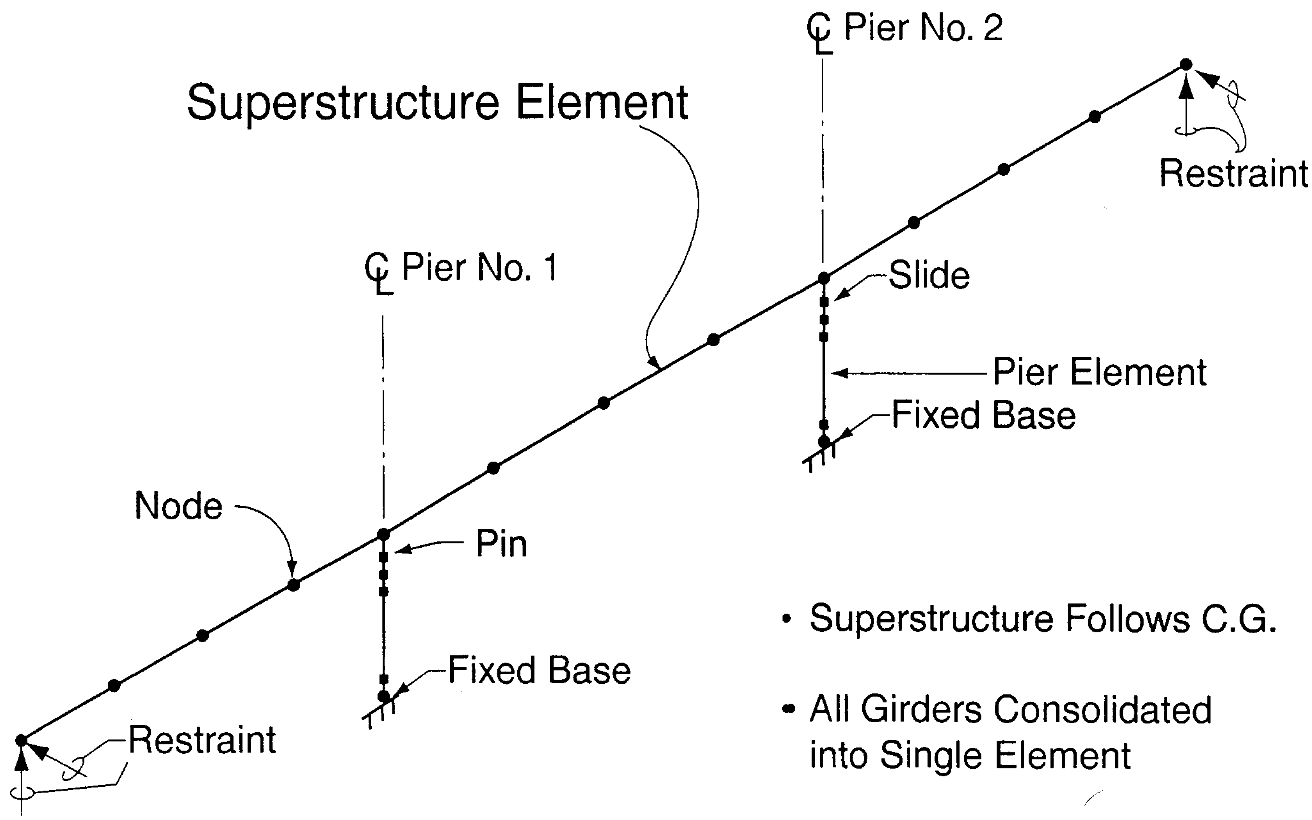


Longitudinal Behavior — One Column Resists Loads



Transverse Behavior — Piers and Abutments Resist Loads

Seismic Analysis Model



- Superstructure Follows C.G.
- All Girders Consolidated into Single Element

Superstructure Properties

Location	Area	Effective Density	Moment of Inertia		
			Bending in Horiz. Plane	Bending in Vert. Plane	
	A (ft ^{2})	γ [a] (k/ft ^{3})	I horiz[b] (ft ^{4})	y bar [c] (ft)	I vert[b] (ft ^{4})
Abutment	81.0	0.166	36207	1.377	296
End Span 1/4 Pt	81.0	0.166	36207	1.377	296
1/2 Pt	81.0	0.166	36353	1.407	311
3/4 Pt	84.3	0.162	36607	1.698	473
Pier	104.0	0.143	45988	2.477	996
Center Span 1/4 Pt	83.4	0.163	37206	1.603	417
1/2 Pt	81.0	0.166	36207	1.377	296

a. Includes Weight of Barriers, Overlay, Forms, Stiffeners, and Cross Frames

b. I Based on Full Composite Action of Deck and Girders

c. 'y bar' Is Measured from the Top of the 9 in. Deck

Superstructure Specifics

- Properties Based on Equivalent Concrete
- Weights Include

  Concrete $w_c = 8.16$ kip/ft

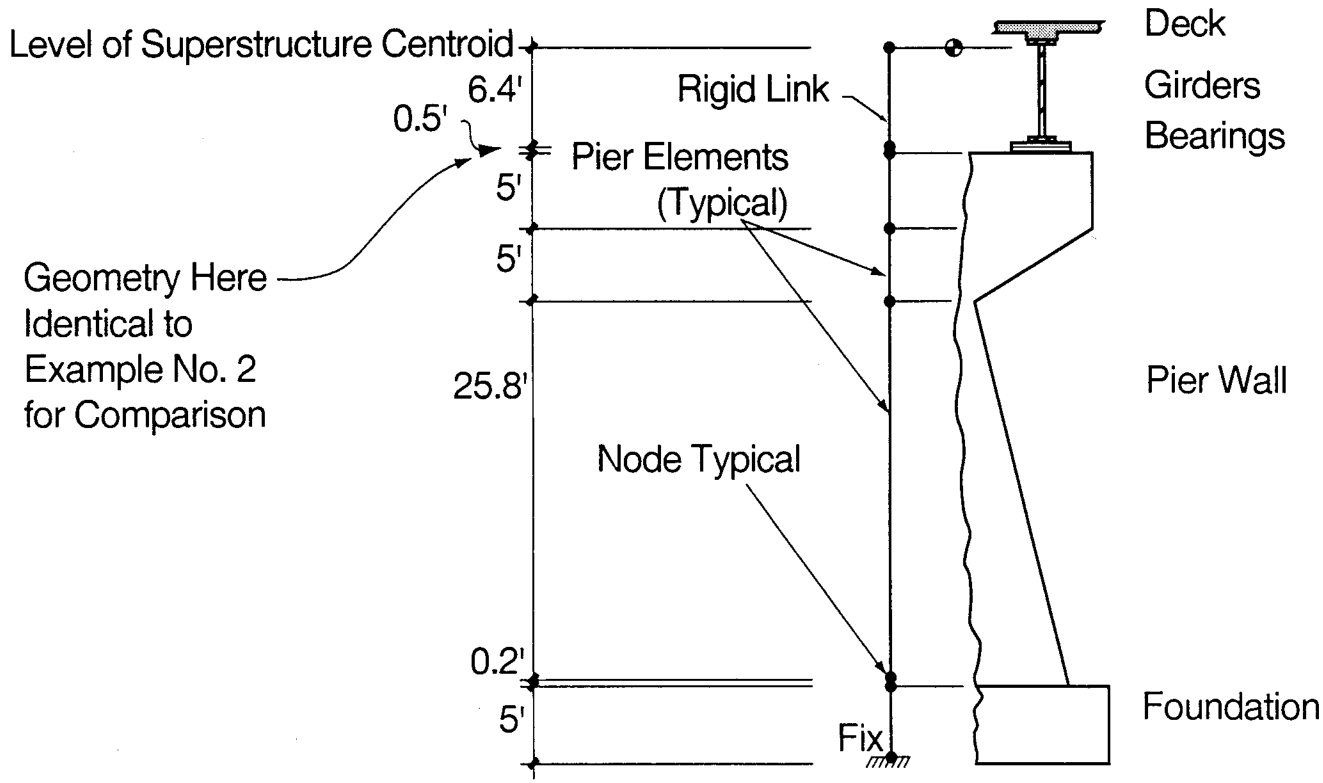
  Girders $w_g = 3.04$ kip/ft to 1.63 kip/ft

  Barrier Overlay, Stay-in-Place Forms, Allowance for Cross Frames and Stiffeners

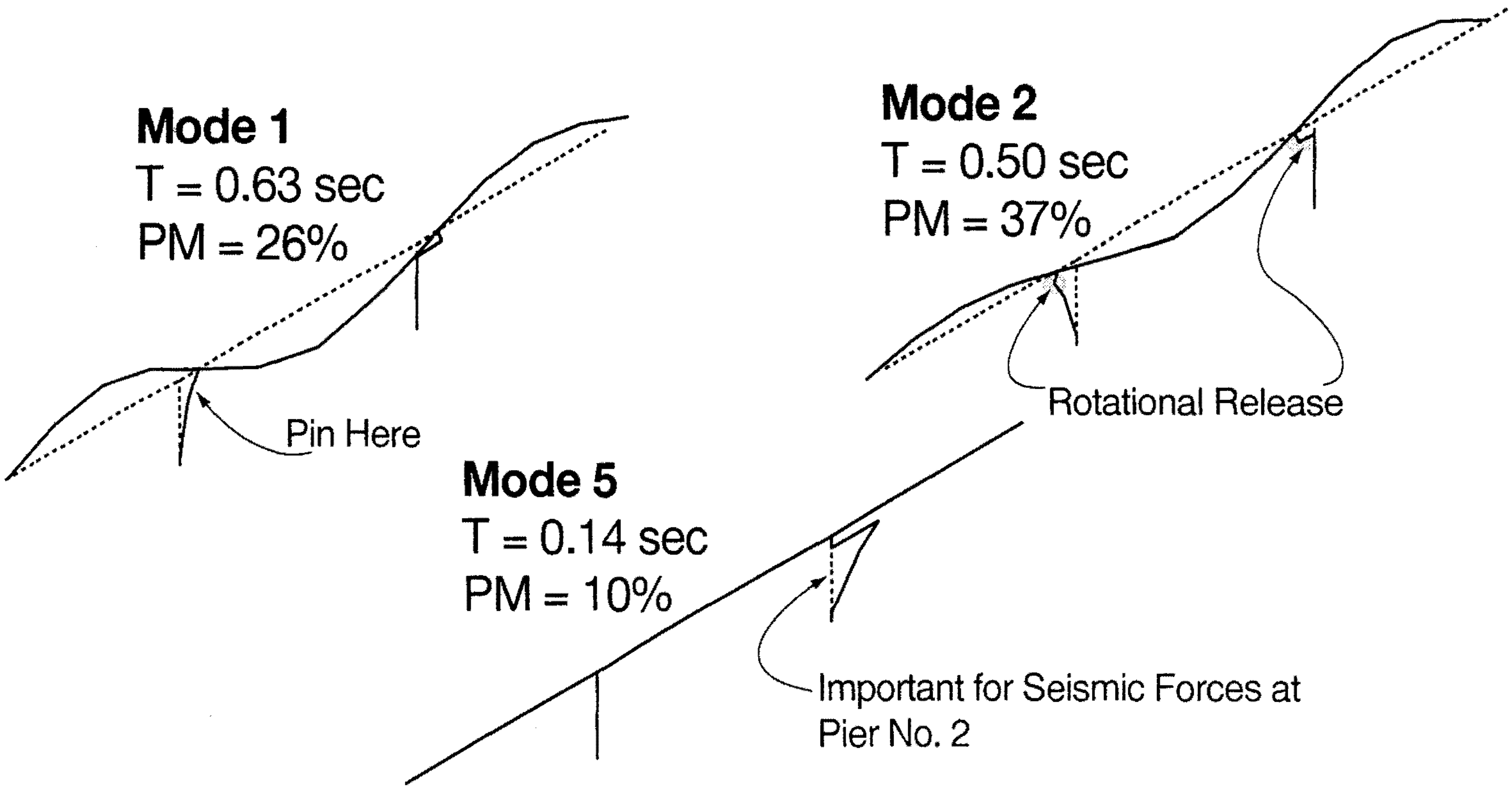
  $$w_m = 3.69 \text{ kip/ft}$$

- Full Composite Action Assumed

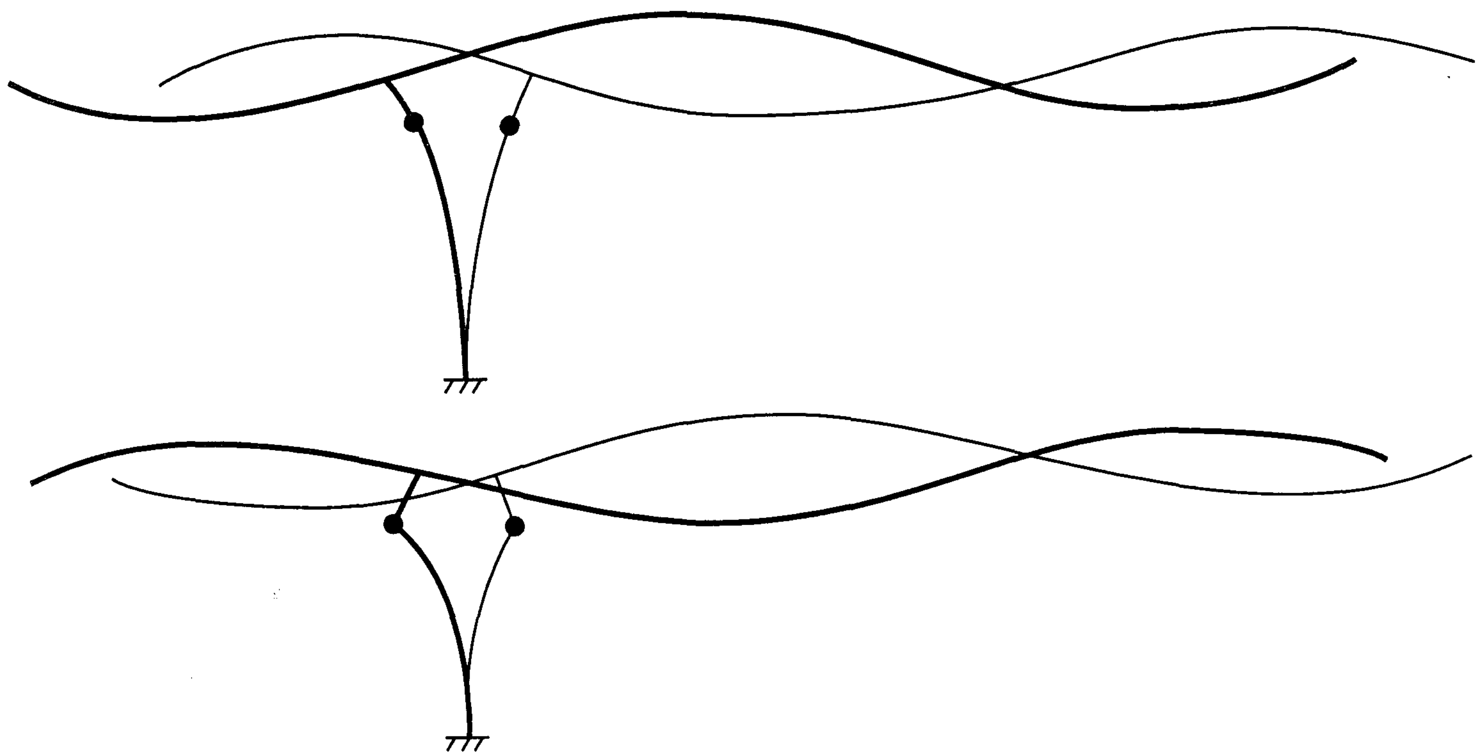
Pier Geometry



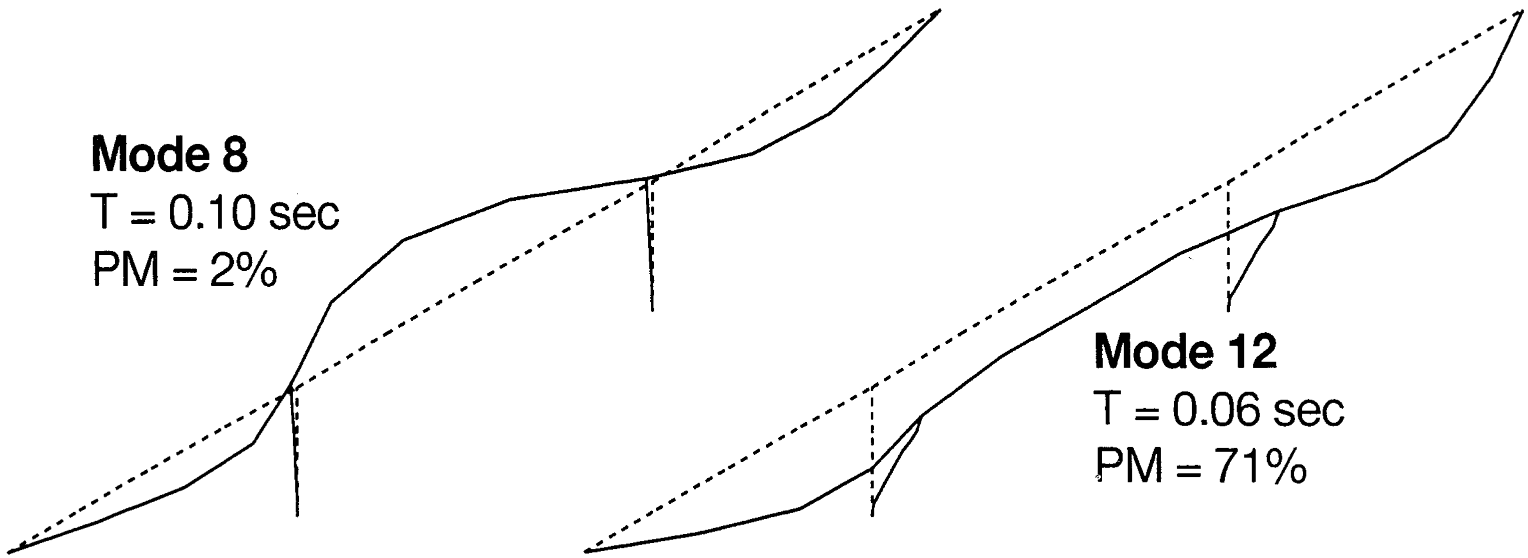
Longitudinal Mode Shapes



Reasons for Two Longitudinal Modes

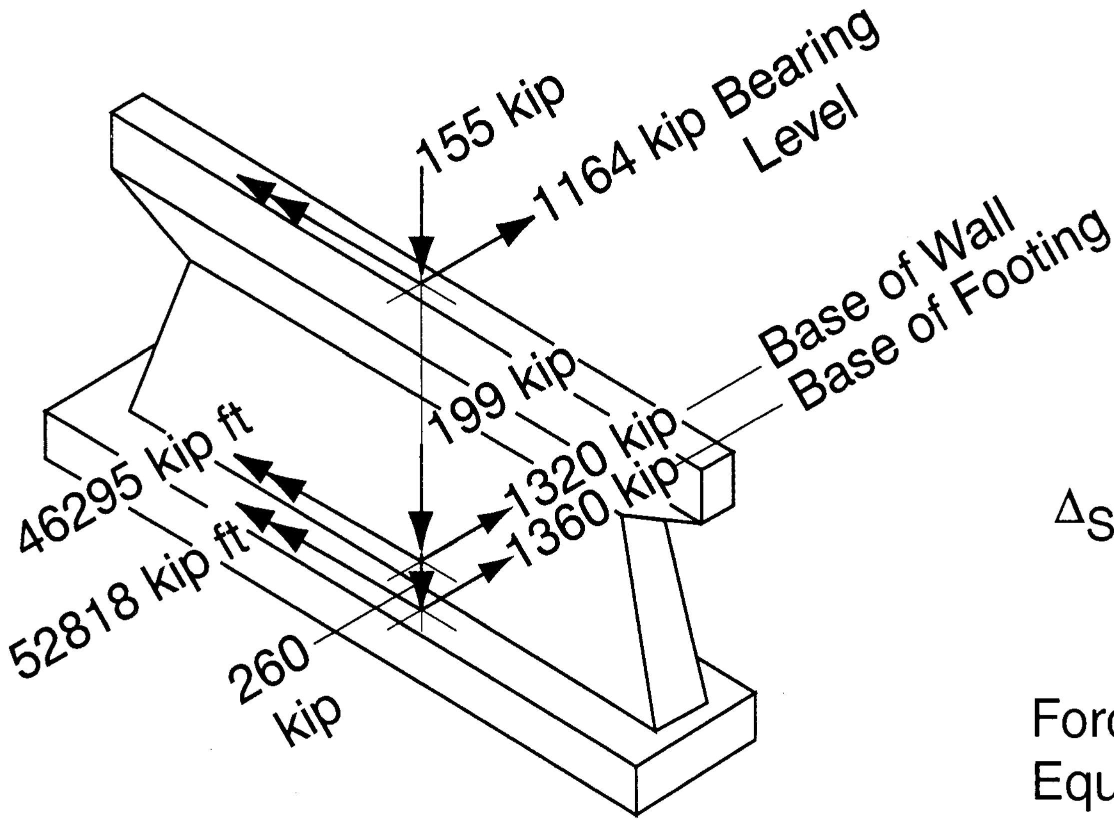


Transverse Mode Shapes



(Recall 3 • No. of Spans = 9)

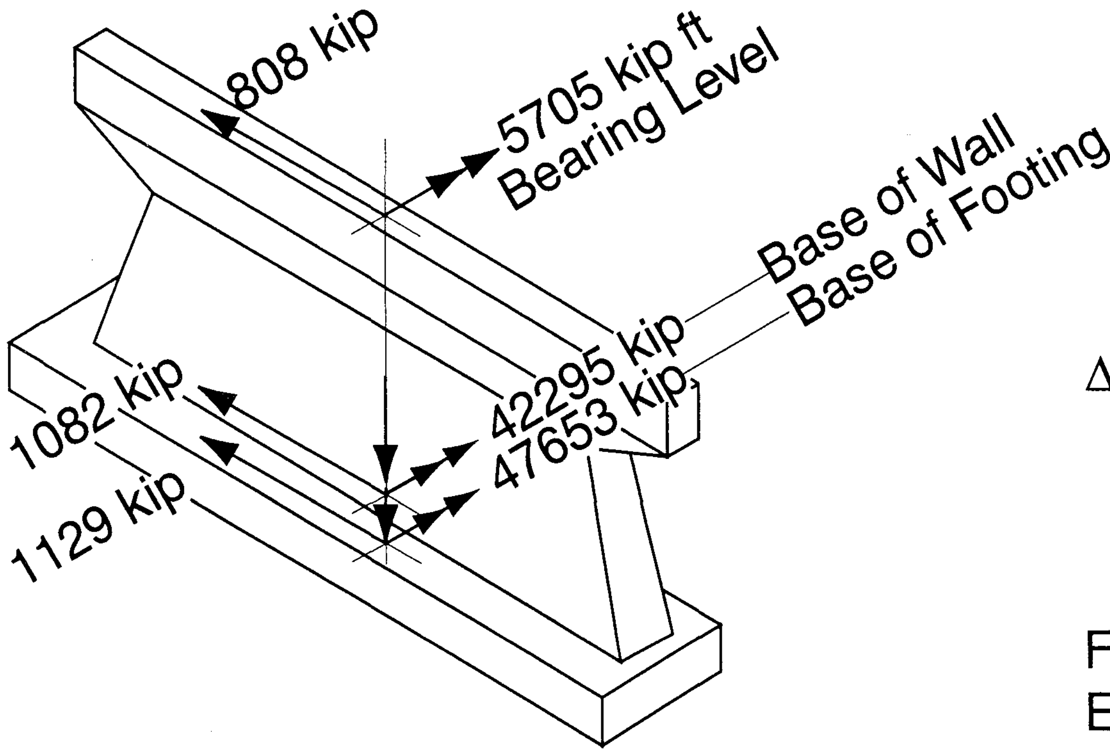
Longitudinal Modal Analysis Results



$\Delta_{Super} = 0.61$ in.

Forces Not Shown Equal Zero

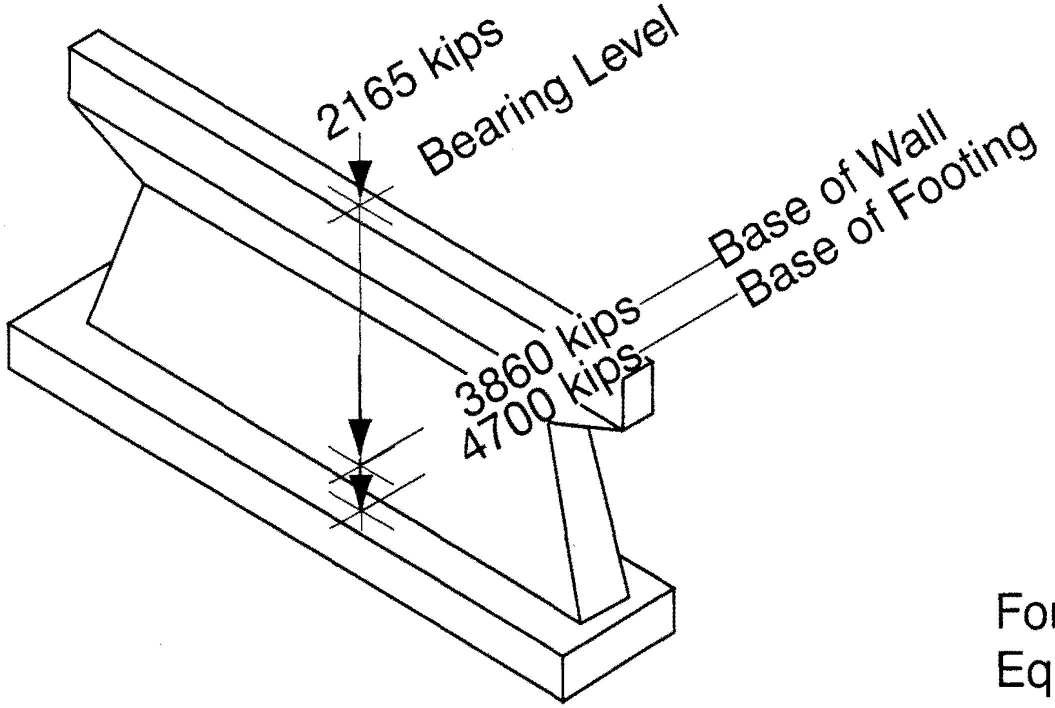
Transverse Modal Analysis Results



$\Delta_{Super} = 0.014$ in.

Forces Not Shown
Equal Zero

Dead Load Analysis Results / Spine Model



Forces Not Shown
Equal Zero

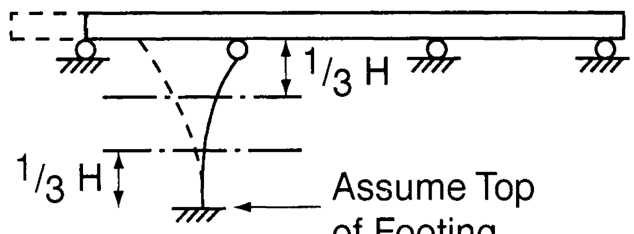
Check of Results / Hand and Computer

Strategy:
- Compare Period and Base Shear

Use:
- Hand Model with Rigid Superstructure
- Computer Model with Rigid Superstructure

  (Only Change from Previous Modal Analysis)

Hand Check



- **Seismic Weight**

  $W_{super} = 5525$ kip
  $W_{1/3} = 517$ kip

  $W_{total} = 6041$ kip

- **Stiffness**

  Use Stiffness at $^1/_3$ of Height of Tapered Wall Above the Footing

B = 60.46 ft T = 5.333 ft

$$K = \frac{3(519000)764}{(36)^3} = 25508 \text{ kip/ft}$$

Hand Check (continued)

- **Period** $T_{Long} = 2\,\pi \sqrt{\dfrac{W}{g\,K}} = 2\,\pi \sqrt{\dfrac{6041}{32.2\ (25508)}}$

  $T_{Long} = 0.54$ sec Bracketed by Mode 1 and 2 Periods

- **Base Shear** $V_{Long} = C_s W = \dfrac{1.2(0.15)1.0}{(0.54)^{2/3}}\ (6041)$

  $V_{Long} = (0.272)(6041) = 1642$ kips

Computer Model with 'Rigid' Superstructure

Let:

$I_{super} \rightsquigarrow 10^7 \cdot I_{super} \rightsquigarrow T_{long} = 0.53 \text{ sec}$

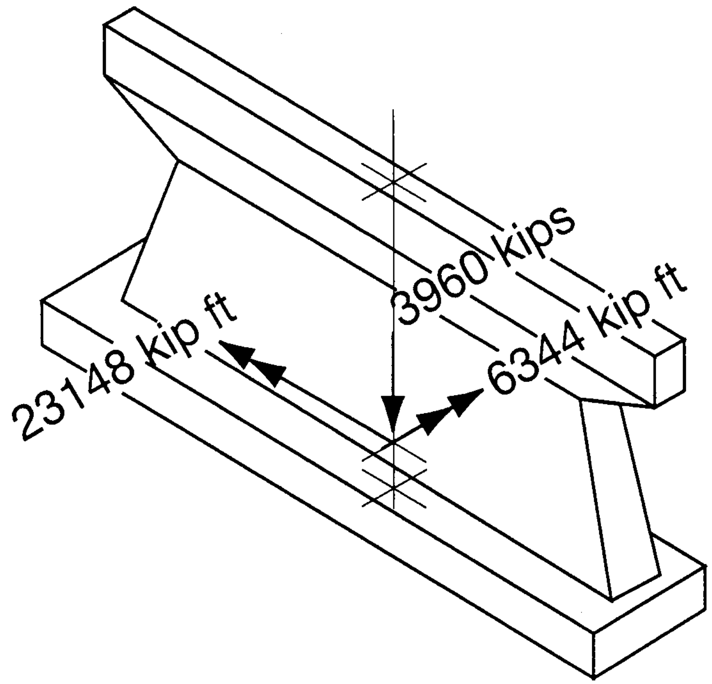
Then:

$V_{long} = 1776 \text{ kip}$

Comparison of Results and Checks

• **Basic Model**	V = 1320 kip at Base of Wall	
• **Hand Check**	V = 1642 kip ...	Higher Due to Single Mode Contributing All Response
• **Rigid Superstructure Computer Model**	V = 1776 kip ...	Higher Than Hand Check Due to Contribution of Lower Part of Pier (~ 90 kip)

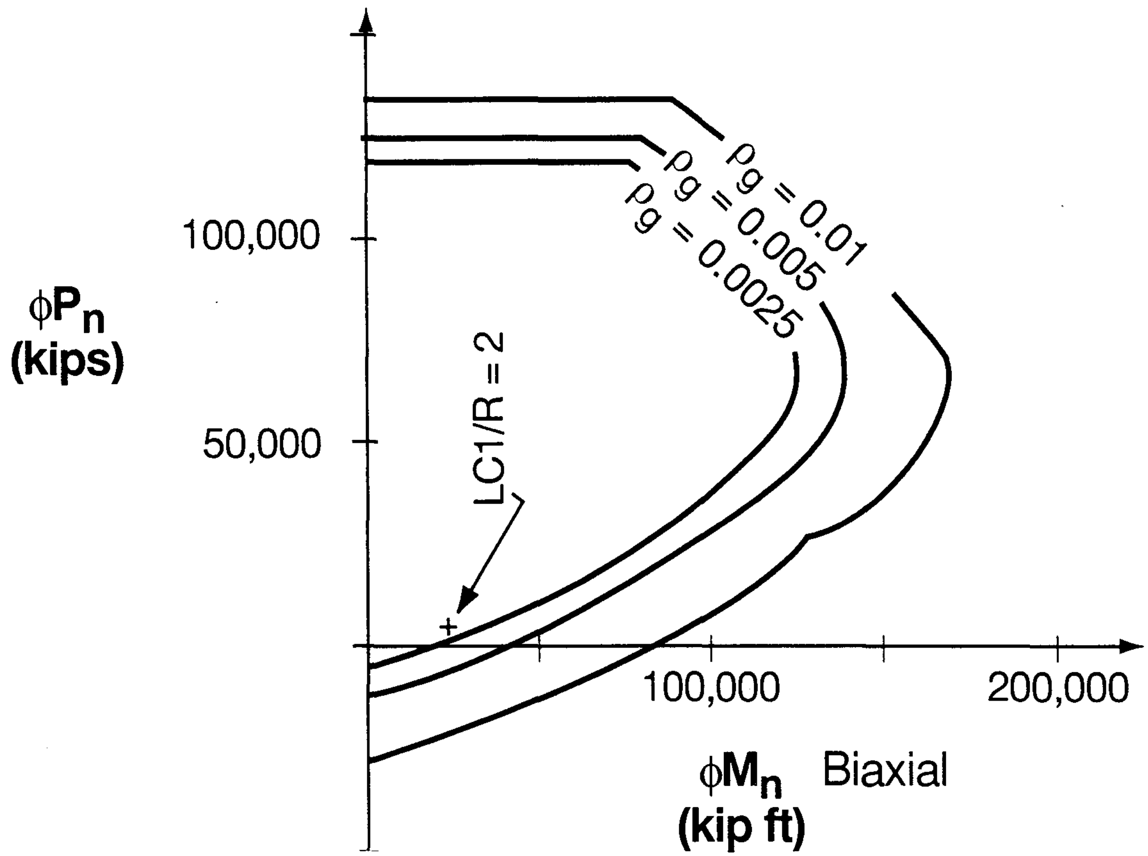
Design Forces at Base of Wall



R = 2 Weak

R = 2 Strong

Vertical Reinforcement Options



Minimum Vertical Steel Considerations

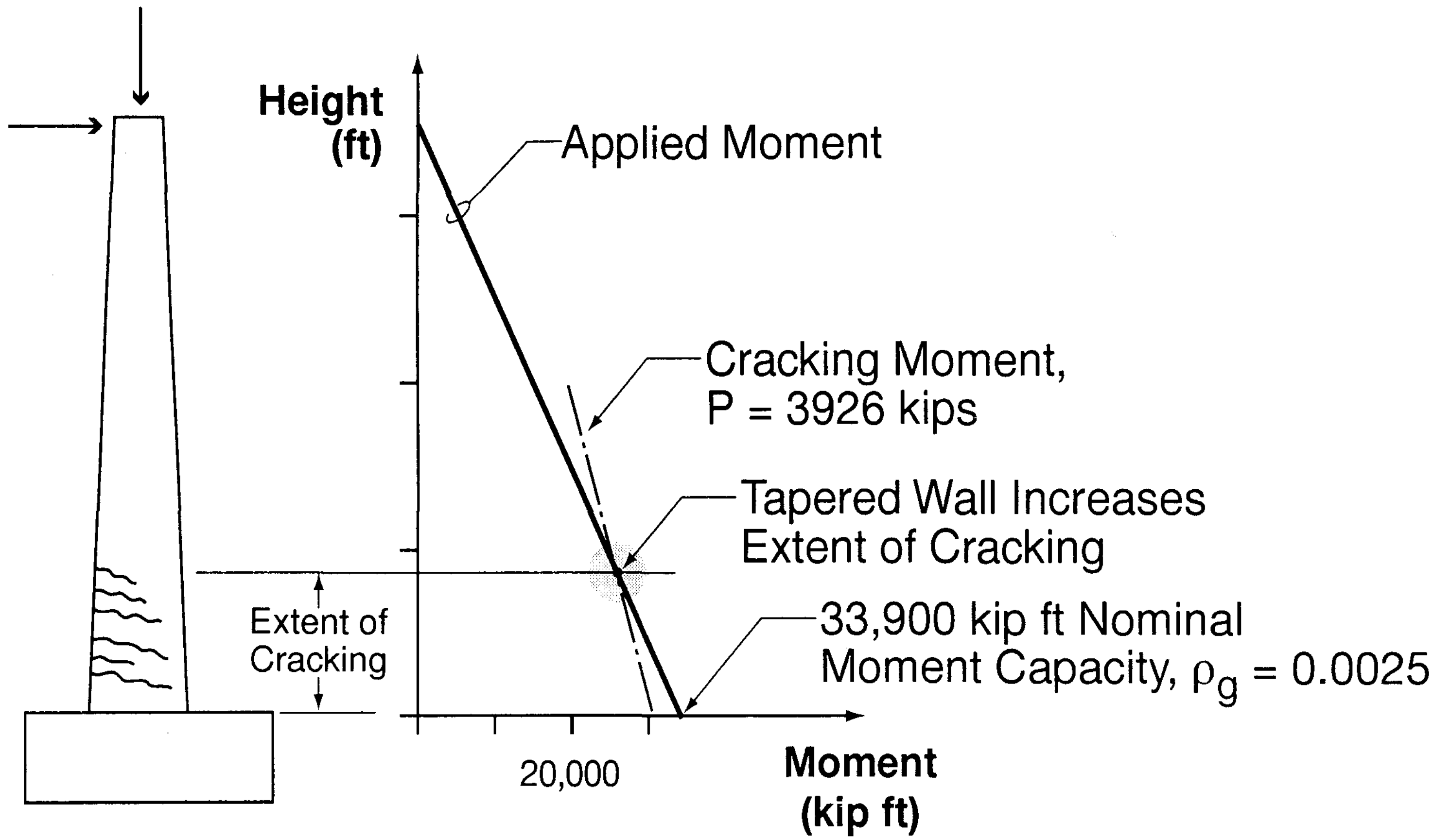
a) Wall — $\rho_g \geq 0.0025$ SPC C & D Div. I-A. 7.6.3

b) $\phi M_n \geq 1.2\ M_{crack}$ (Flexural Members) Div. I 8.17.1.1

This Wall:

- $\rho_g = 0.0025$ Can Satisfy a) Since R = 2
- Consider b) for Crack Distribution

Distribution of Cracking

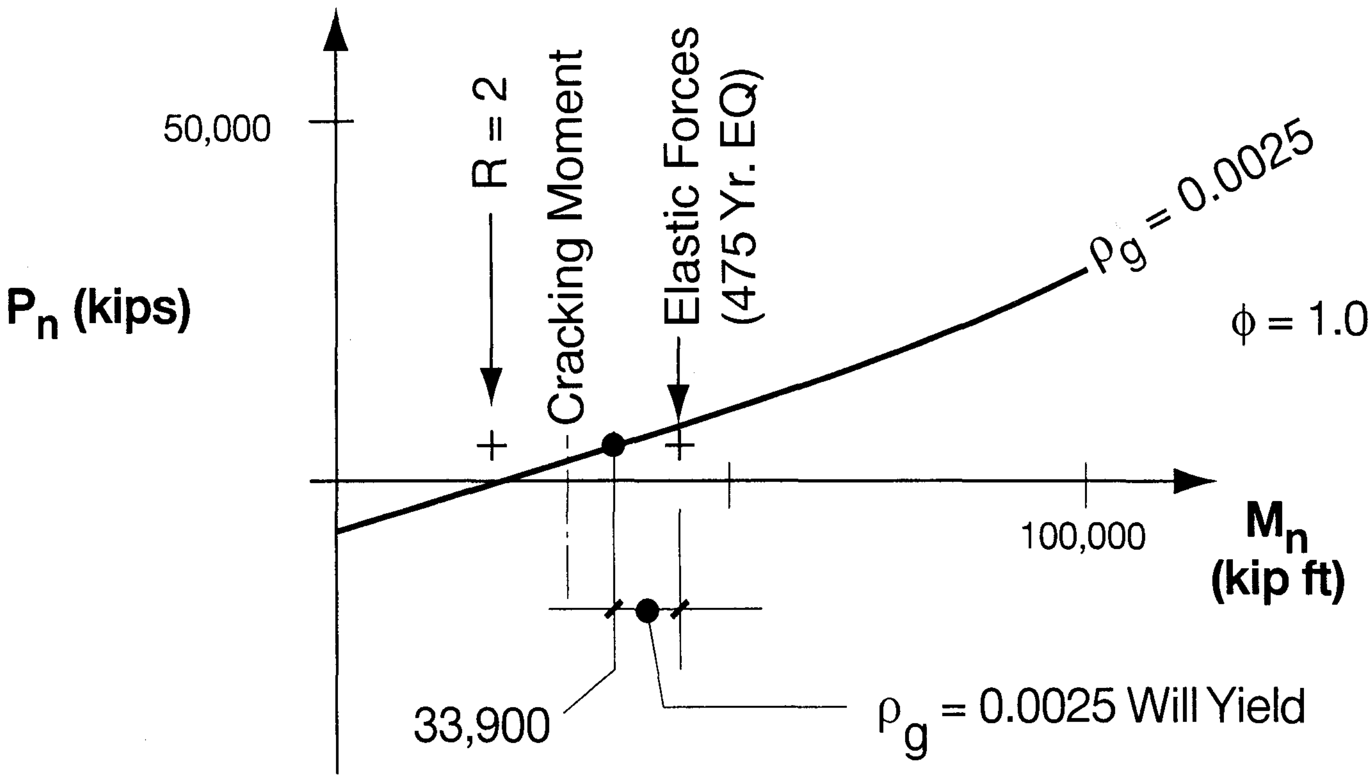


Selection of Vertical Reinforcement

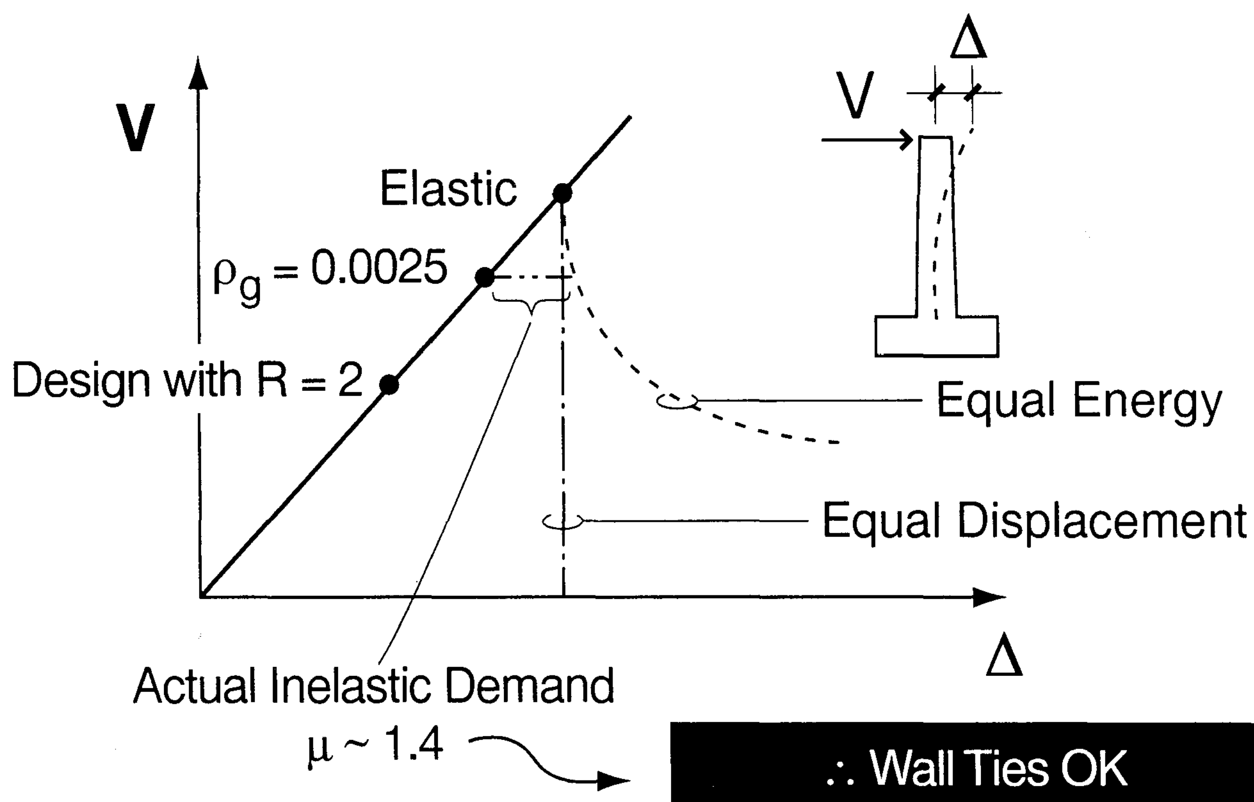
Use $\rho_g = 0.0025 \longrightarrow$ 142 #9 Bars

- This Will Work for R = 2
- Wall Is Expected to Yield During 475 Year Earthquake, but Ductility Demand Will Be Low ($M_{elas} \sim 1.2\ M_n$)
- Even Though $M_n \sim 1.10\ M_{cr}$, Cracking Will Be Distributed Due to Wall Taper

Nominal Capacity of Wall in Weak Direction



Expected Inelastic Demands



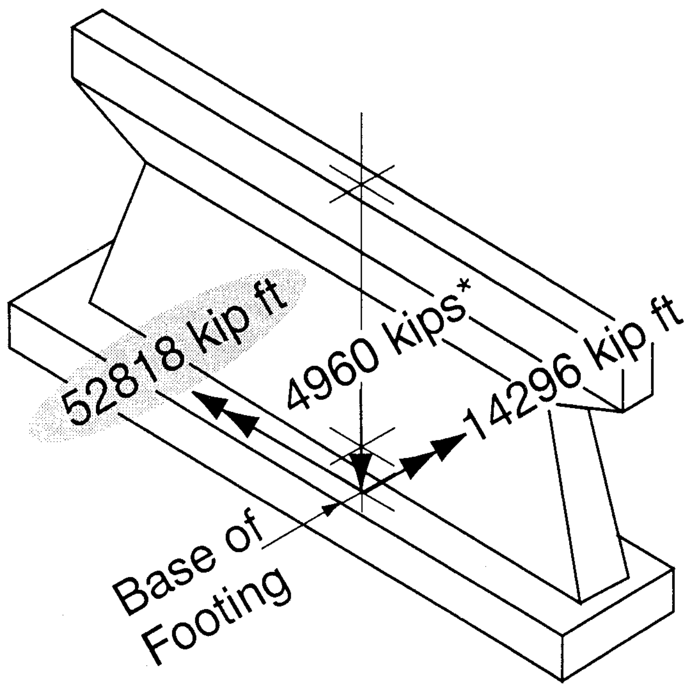
∴ Wall Ties OK

Wall Cross Ties

Weak Direction / Designed as a Column / R = 2

Use #4 at 2 ft O.C. Horizontial and 8 in. Vertical
See Design Example No. 2

Foundation Design Forces / Controlling Case LC1



R = $^{2}/_{2}$ Weak

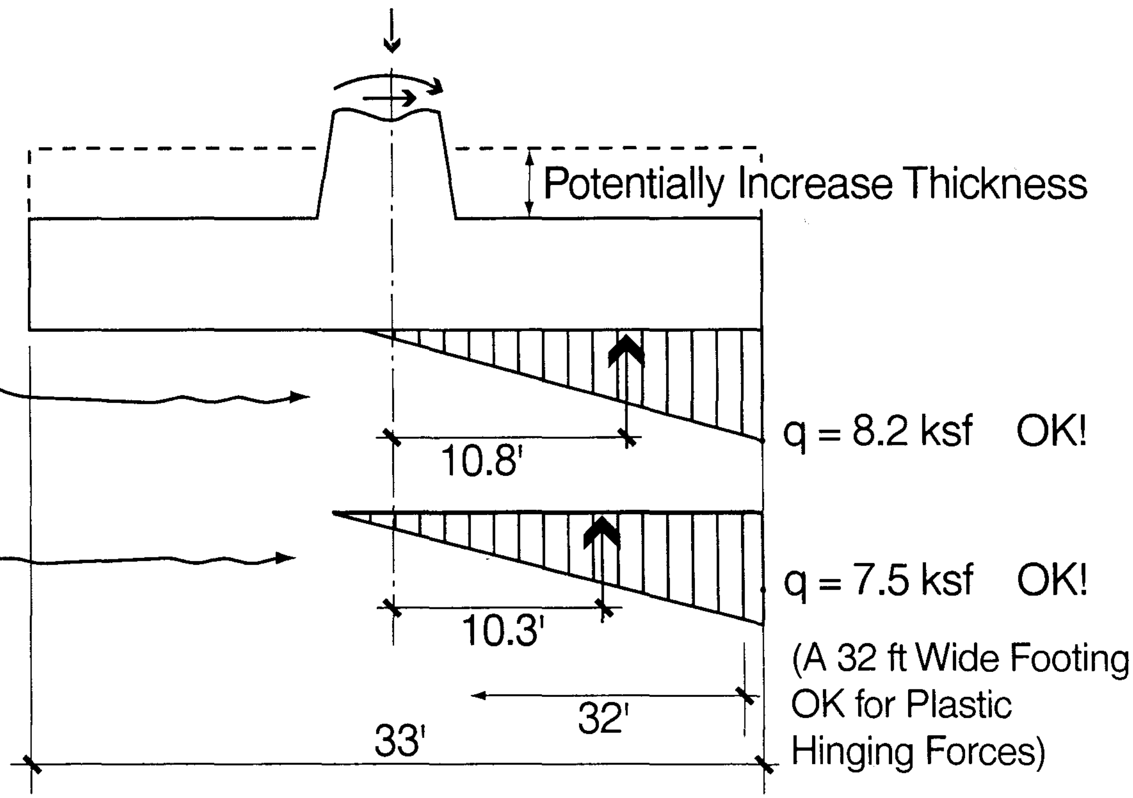
R = $^{2}/_{2}$ Strong

*Does Not Include Buoyancy and Stone Fill

Foundation Behavior / 33' Footing

Results for:

- Design Forces
  $R = {}^{2}/_{2}$
  (Elastic)
- Plastic Hinging
  Forces
  $\rho_g = 0.0025$



Choices and Implications / Flexural Design

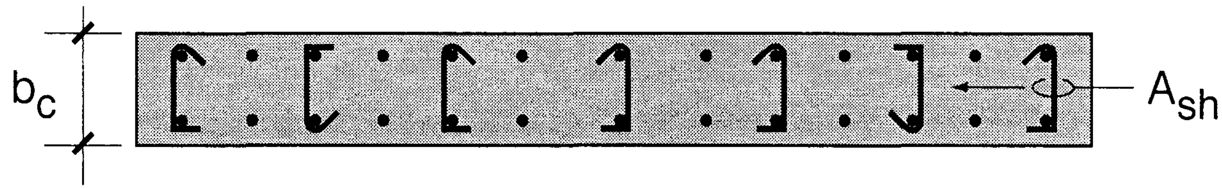
SPC B Weak Direction

	R = 2 (Wall)	R = 3 (Column) 1% Vertical Steel
Wall:	Less Vertical Steel (ρ_g = 0.0025)	More Cross Ties in Hinge Zone
Foundation:	Larger Footing	Smaller Footing

Cross Ties

Weak Direction / Designed as a Column / R = 3

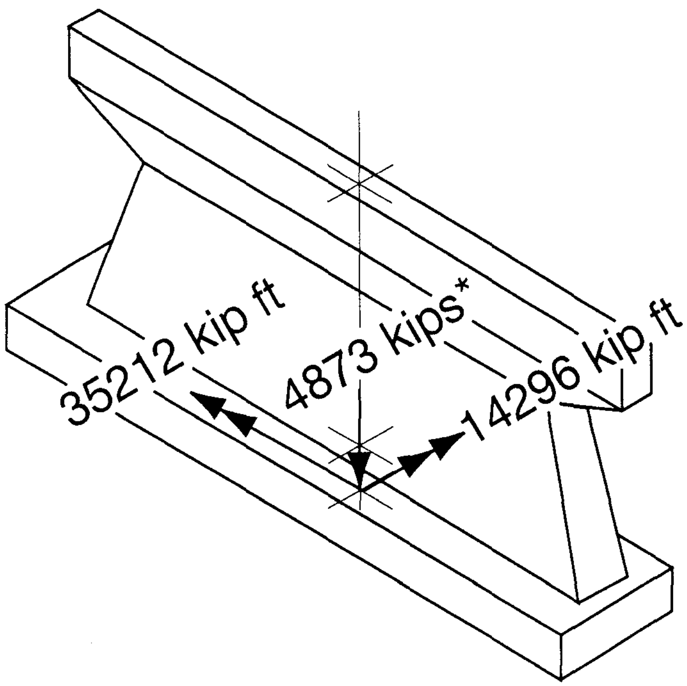
I-A / 6.6.2 $A_{sh} = 0.3ah_c \left[\frac{A_g}{A_{core}} - 1 \right] \geq 0.12ah_c \frac{f'_c}{f_{yh}}$



Try #7 ⟶ 63 Required / Use 67 #7 Cross Ties, One for Each Vertical, at 6 in. Vertical Spacing

Cross Ties Required Over Lower 6 ft ~ Plastic Hinge Zone

Foundation Design Forces



Design as a Column

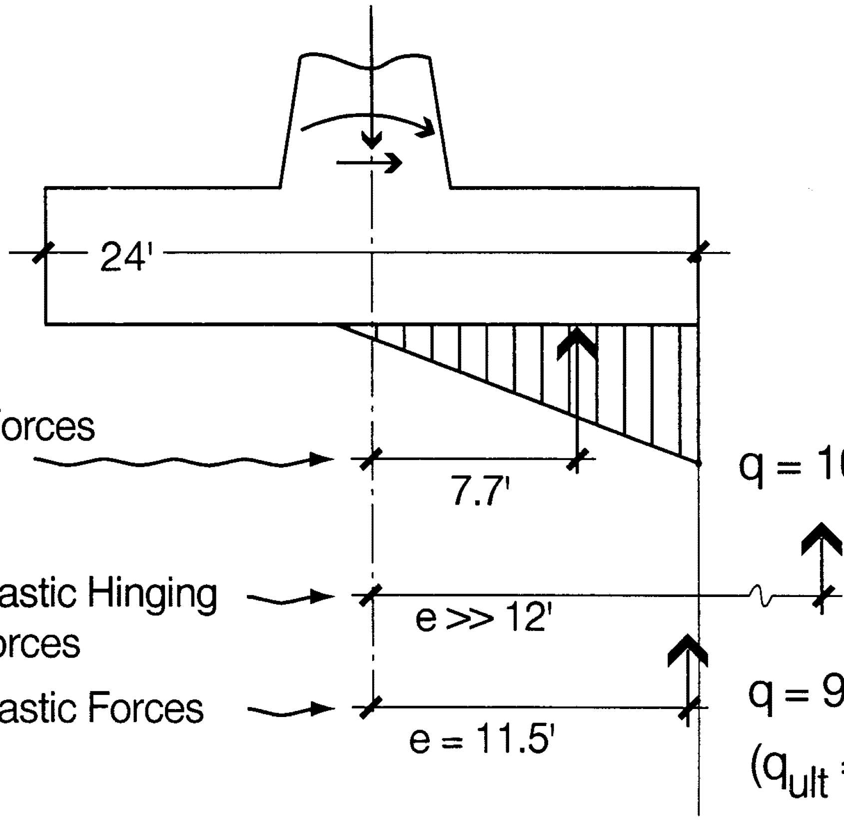
R = $^3/_2$ Weak

R = 1 Strong

*Does Not Include Buoyancy and Stone Fill

Foundation Behavior / 24' Footing

Results for:



- Design Forces
  $R = 3/2$ — $q = 10.1$ ksf — OK!

Not Required in SPC B:

- Plastic Hinging Forces — N.G.
- Elastic Forces — $q = 94$ ksf ($q_{ult} = 50$ ksf) — N.G.

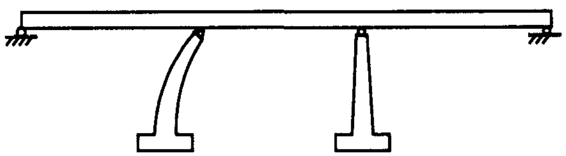
Summary

	Designed As: Wall	Column
Vertical Reinforcement	10 Tons	40 Tons ($\rho_g = 0.01$)
Cross Ties	0.6 Tons	4.6 Tons
Footing Width	33 ft	24 ft*

* Permitted by Code for SPC B, But if Designed for Elastic or Hinging Forces 33 ft Would Be Required

Choices and Implications

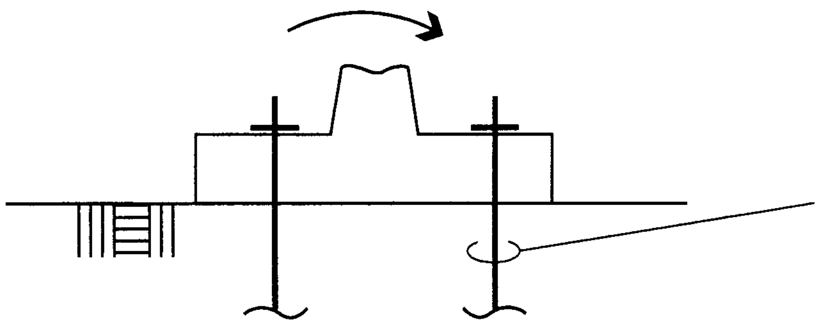
1. Use 33 ft Footing ... Design as a Wall



- Best Solution for Single Conventional Bearing Configuration
- No Foundation Damage

Alternative

2. Use 16 ft Footing … Use Rock Anchors to Prevent Overturning



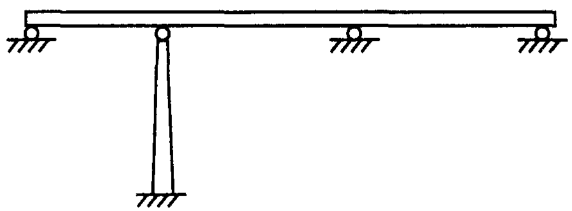
1-3/8 in. ϕ Dywidag
17 Each Side = 34 Total

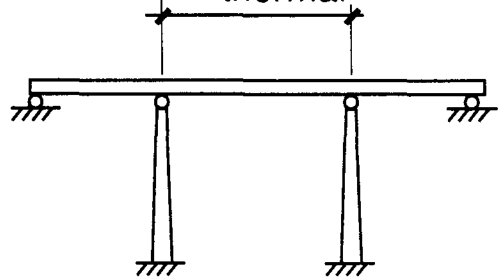
Session 3
Conceptual Design Considerations

- **Conventional vs. Elastomeric Bearings**
- **Longitudinal Releases and Restraints**

Conventional Bearings

$\Delta_{thermal} \approx 1.2'' =$ Problem





$T = 0.52$ sec

$\Delta = 0.74$ in.

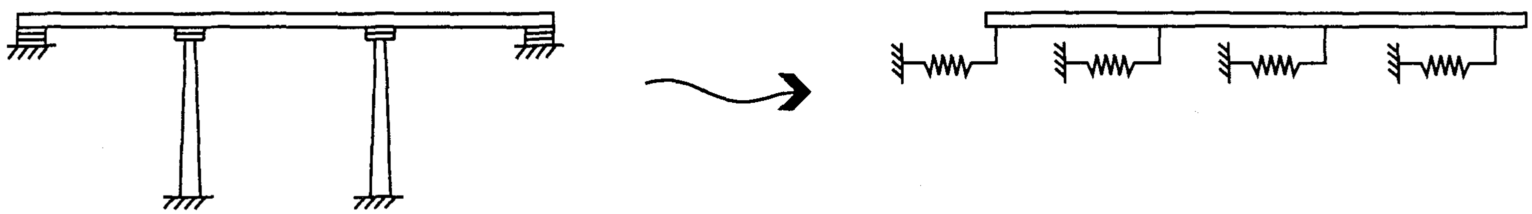
One Restraint

$$T = \frac{0.52}{\sqrt{2}} = 0.37 \text{ sec}$$

$$\Delta = 0.74 \text{ in.} \left(\frac{1}{\left(\frac{1}{\sqrt{2}} \right)^{2/3}} \right) \frac{1}{\underset{\uparrow \text{ No. of Piers}}{2}} = 0.47 \text{ in.}$$

Two Restraints

Elastomeric Pads at Each Support

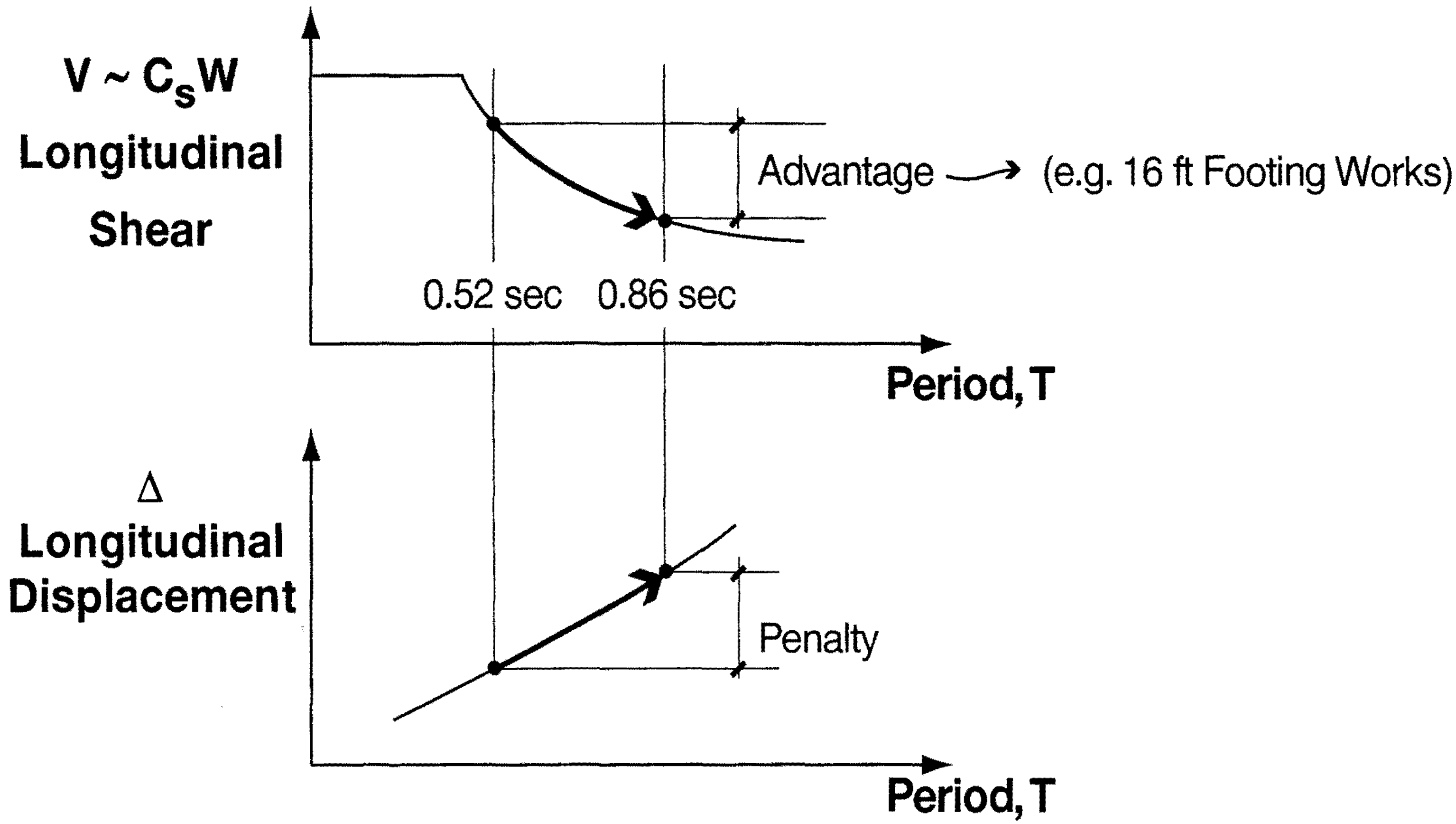


Incorporate
Flexibility of
Elastomeric Pads

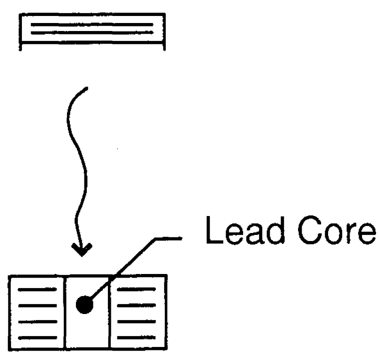
$T \rightarrow 0.86$ sec

$\Delta \rightarrow 1.44$ in.

How the Elastomeric Pads Affect the System

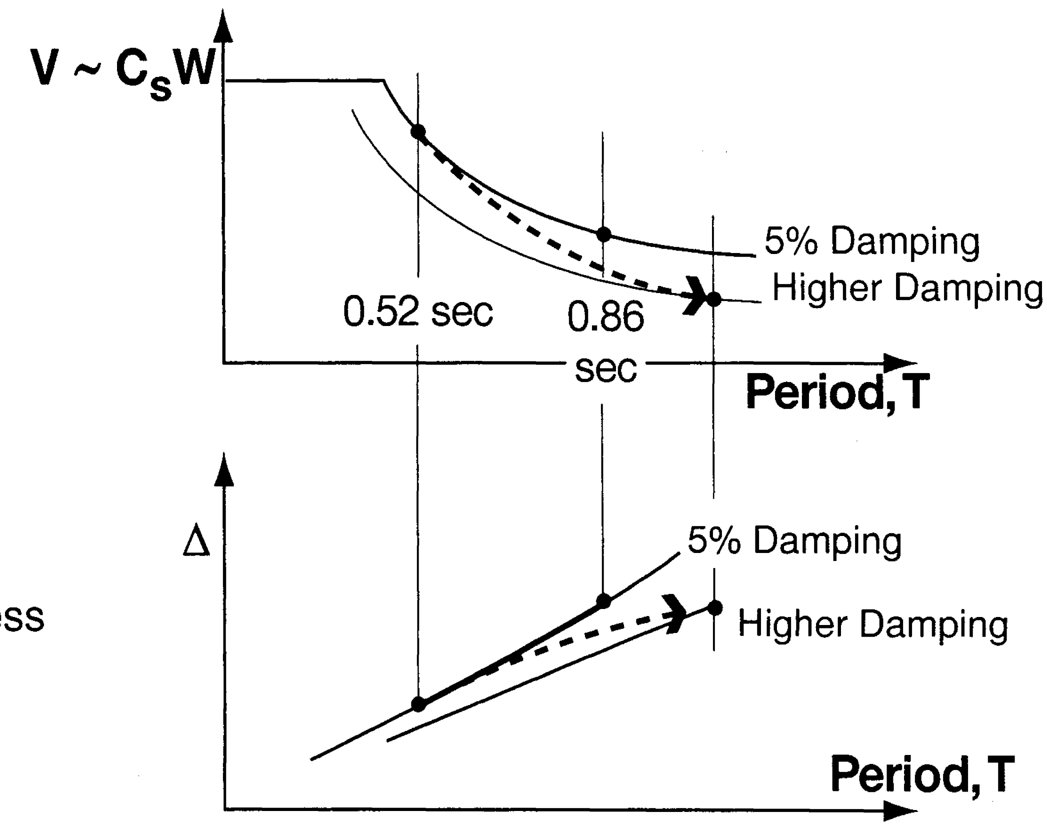


How a 'Base Isolated' Concept Would Affect System



- **Lead Core**
  Damping
  Low Amplitude Stiffness

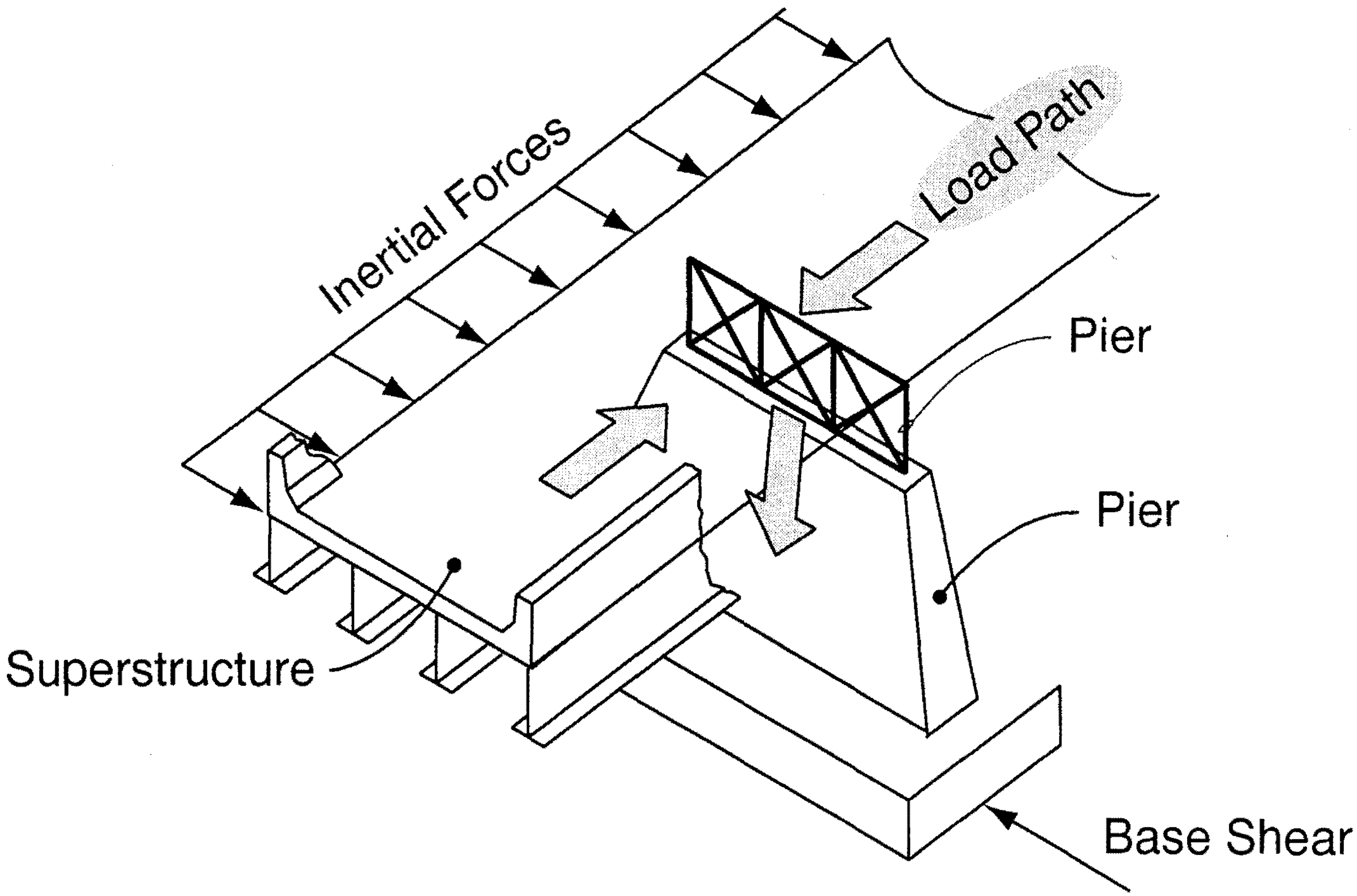
- **Increased Height**
  Added Flexibility



Session 3
Steel Superstructure Issues

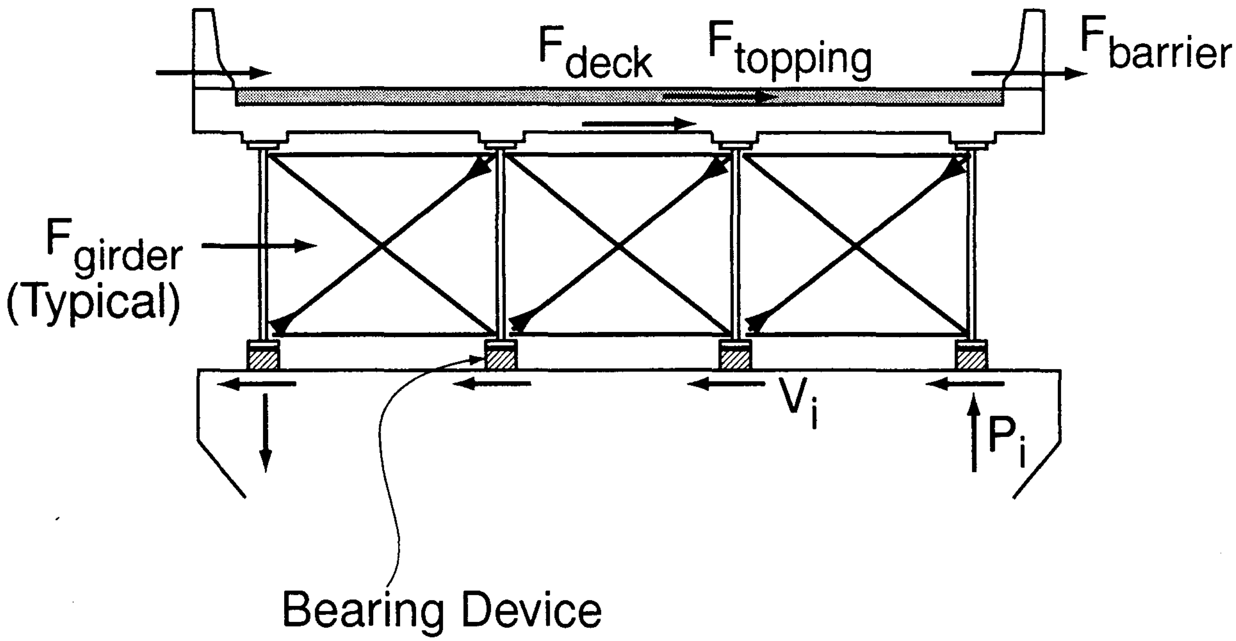
- Cross Frame Design
- Shear Key Considerations

Inertial Forces and Lateral Load Path

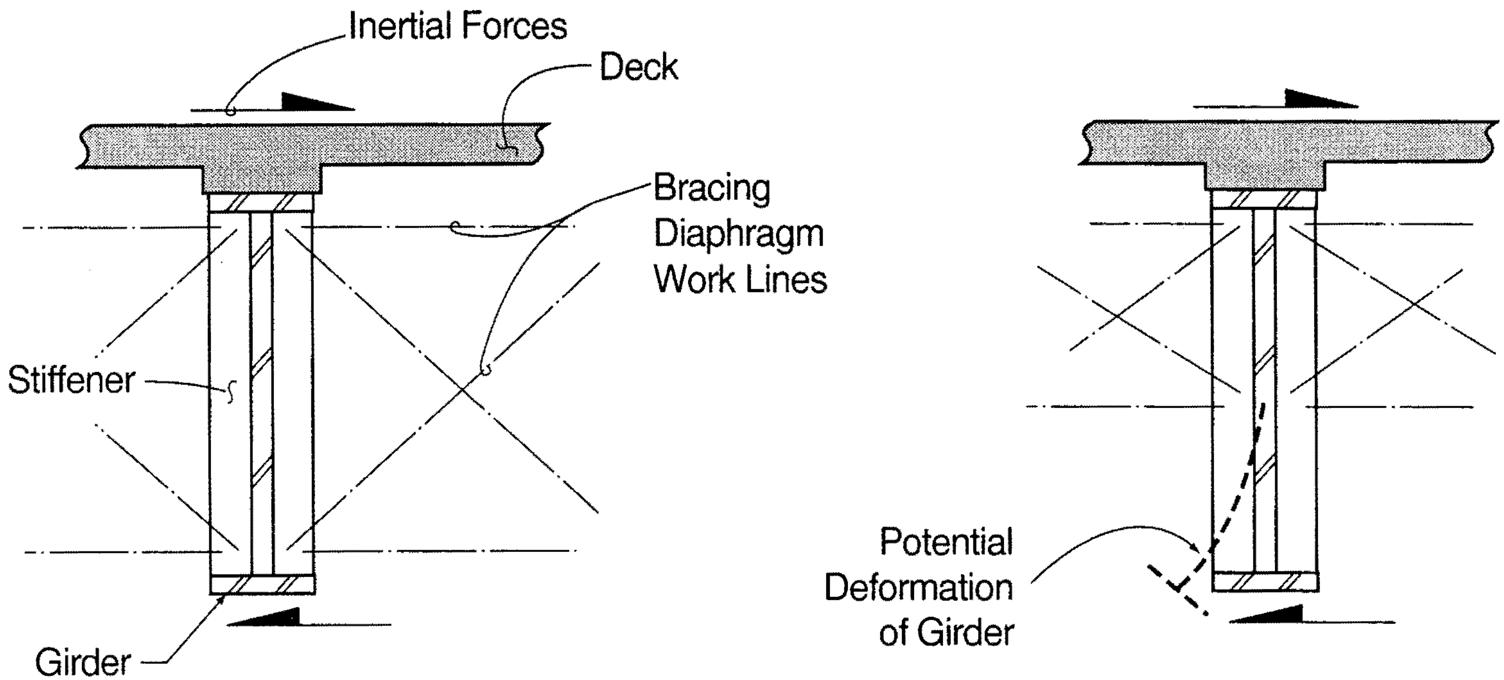


Cross Frame Forces

Pier / Cross Frame / Superstructure



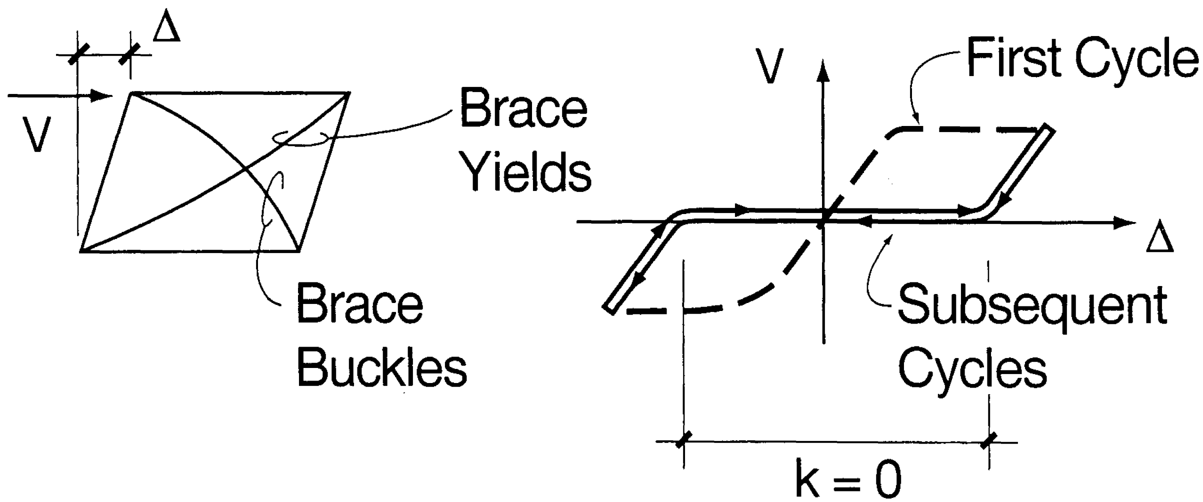
Failure Mode / Lateral Bending



Lateral Rigidity vs. Service Load (Fatigue) Performance

Failure Mode / Tensile Yielding

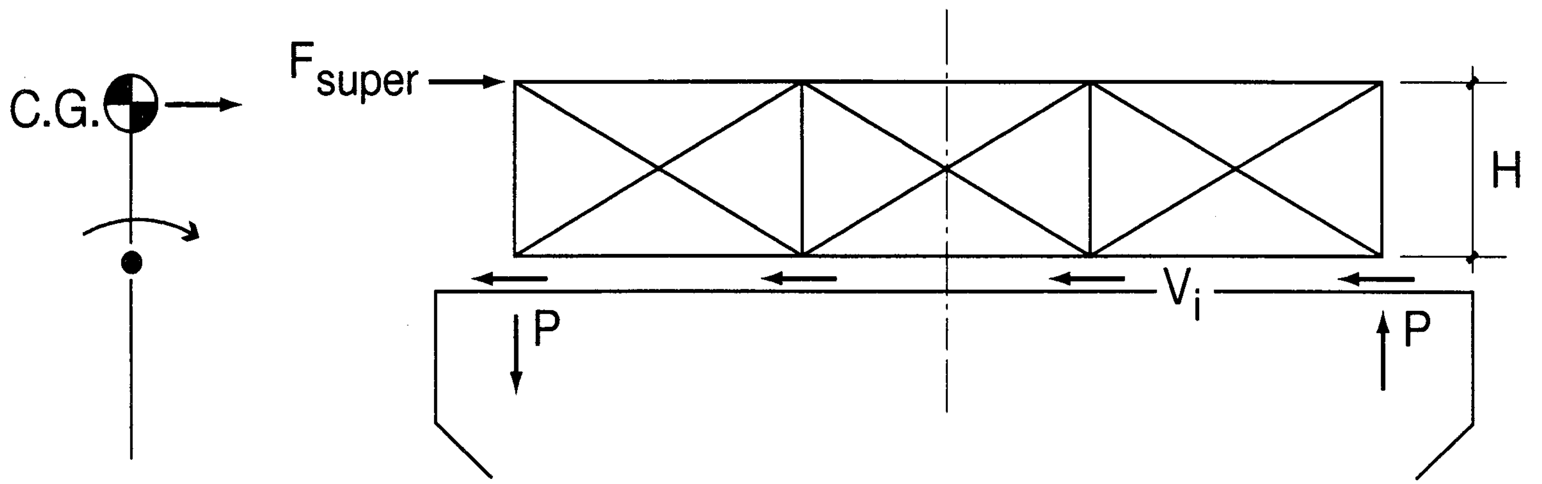
Problem with Cross Frame Yielding



Code Specifies R = 1.0 to Prevent Yielding

- Preserves Elastic (Tight) Response
- Preserves Lateral and Gravity Load Paths

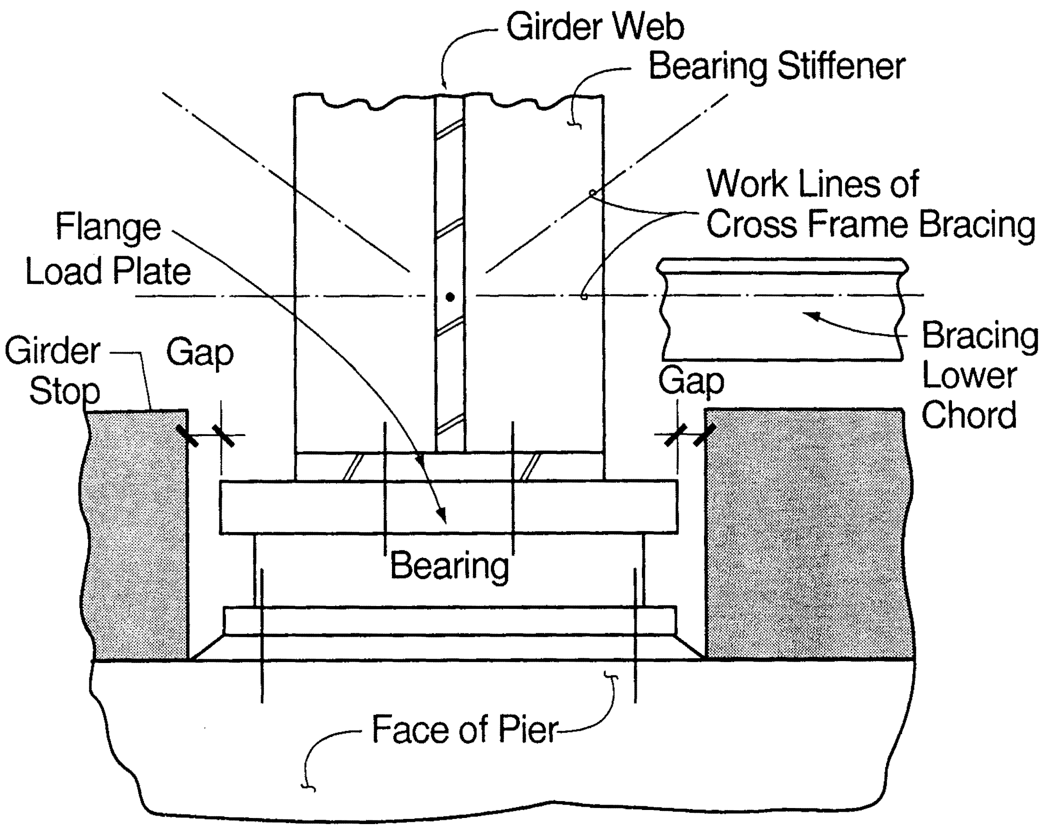
Seismic Model vs. Actual Structure



$$V_i \approx \frac{F_{super}}{n}$$ ... Actual May Be Higher Due to Tolerances

For Relatively Flexible Superstructure Overturning Is Resisted Primarily at Exterior Bearings

Shear Keys / Girder Stops

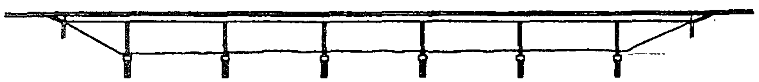


- Failsafe Load Path for Bearing
- Load May Not Be Even Due to Construction Tolerances (Unbuttoning)
- Design to Fail in Ductile Manner

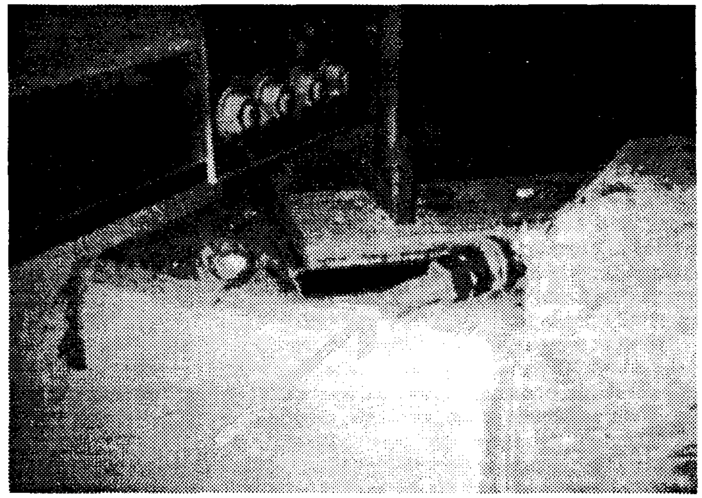
Session 4
Steel Plate Girder Bridge Example
Skew Structure Issues

- Conceptual Behavior
- Stiffness Considerations
- Bearing Orientation and Releases
- Effects on Lateral Behavior

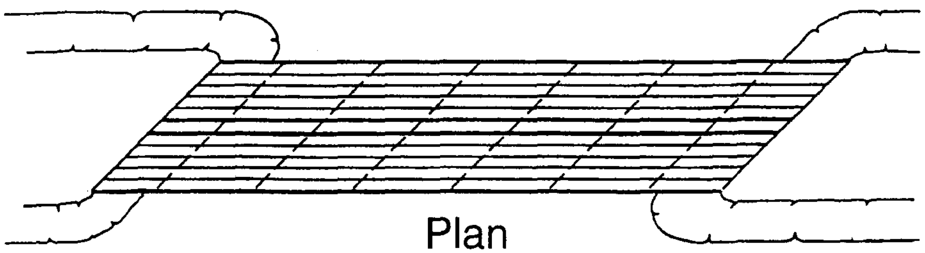
Damage to Steel Superstructure Bridge



Elevation



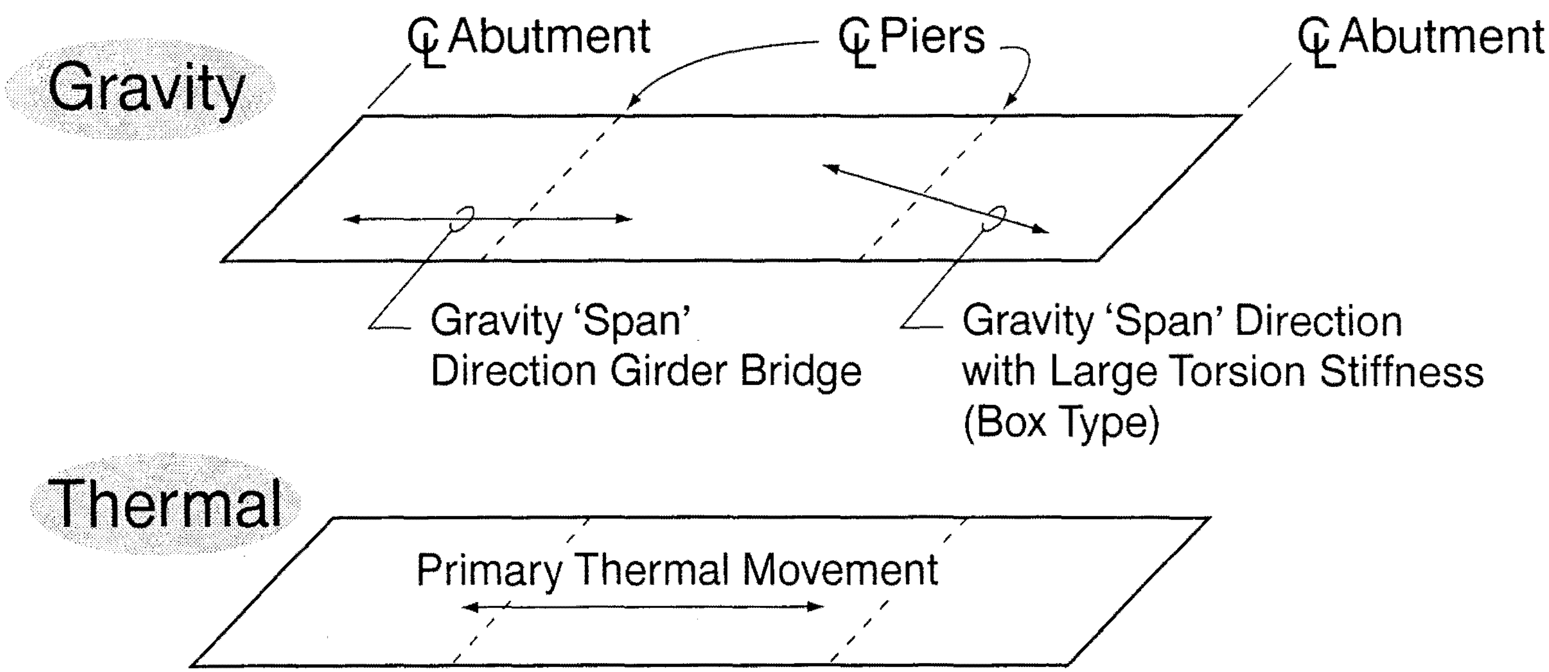
Sheared Anchor Bolts



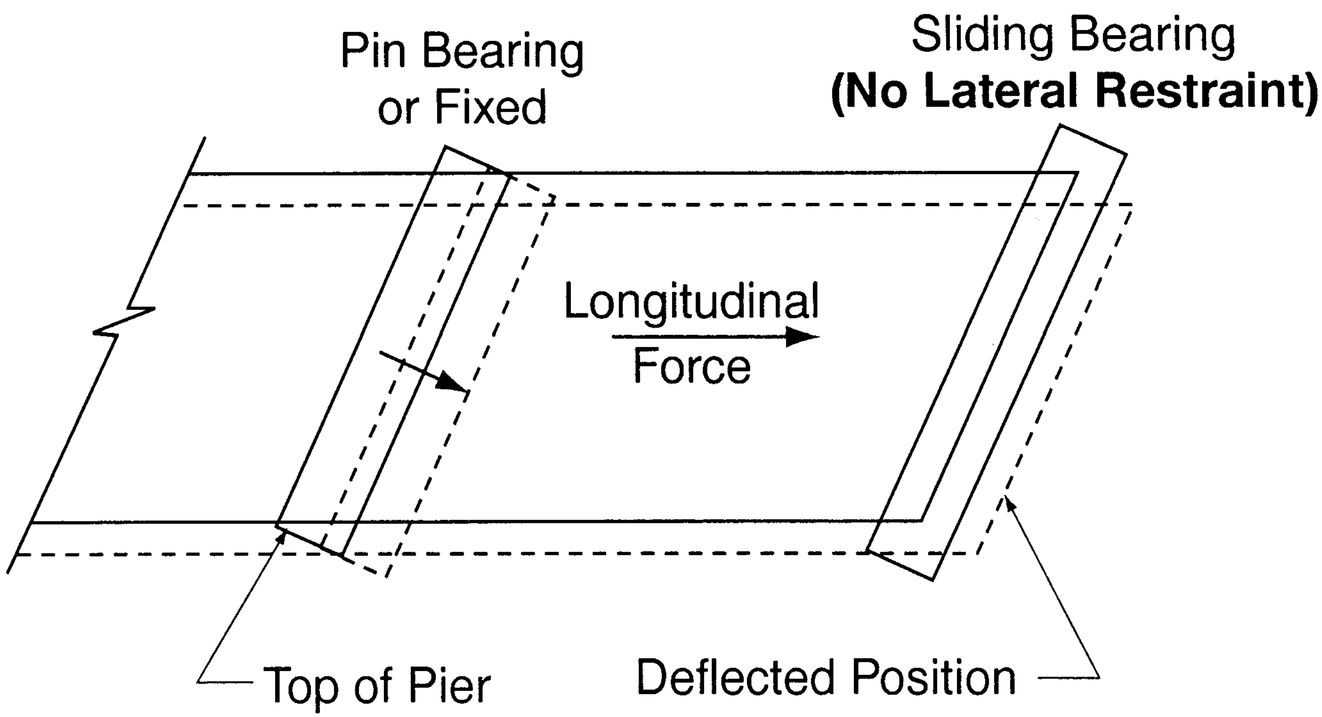
Plan

EERI (1995)

Skew Behavior Under Gravity and Thermal Loads

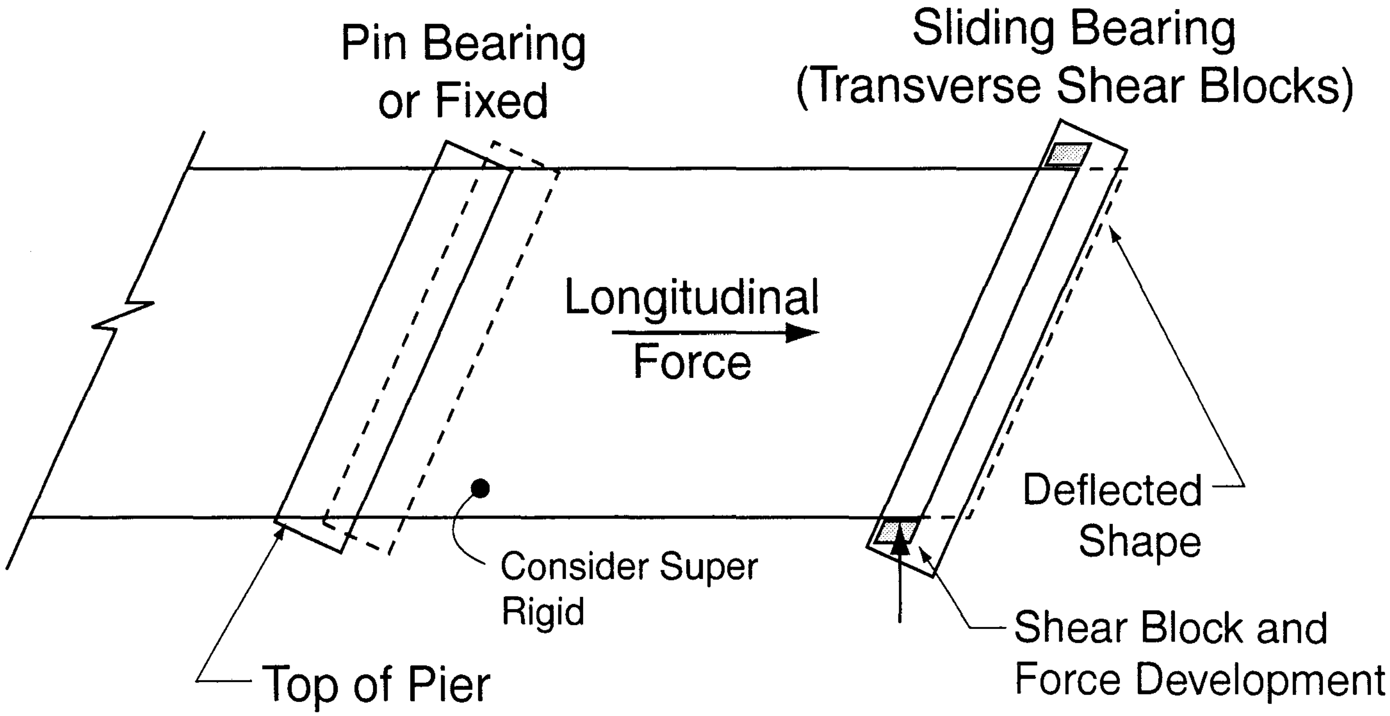


Lateral Loading Concepts



Plan View

Lateral Loading Concepts (continued)



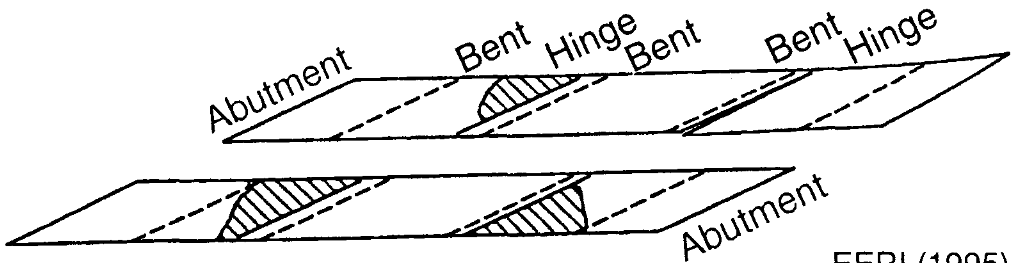
Plan View

Lateral Behavior Observations

- Bridge Would Like to Move Along Weak Axis of Piers
- Shear Blocks Oriented Transversely Prevent Such Movement $\rightsquigarrow$ Large Transverse Forces?
- Behavior Coupled in Orthogonal Plan Directions

  $F_{long} \rightsquigarrow F_{trans}$ and $F_{trans} \rightsquigarrow F_{long}$
- Twisting Also Likely if Mass and Stiffness Centers Are Not Coincident

Damage to Skewed Box Girder Bridge

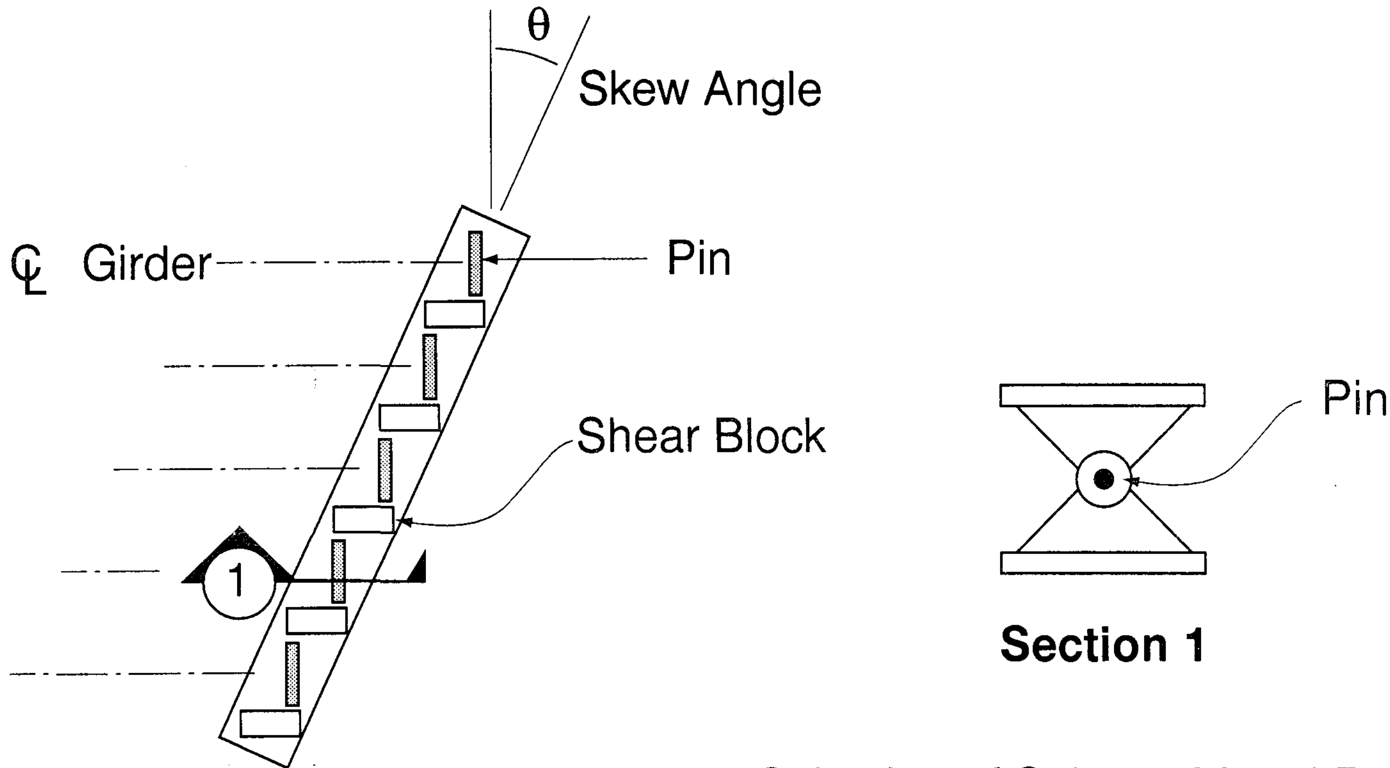


EERI (1995)

USCD (1994)

- End Spans Have Large Eccentricity Between C.M. and C.S.

Steel Superstructure Bearing Orientation

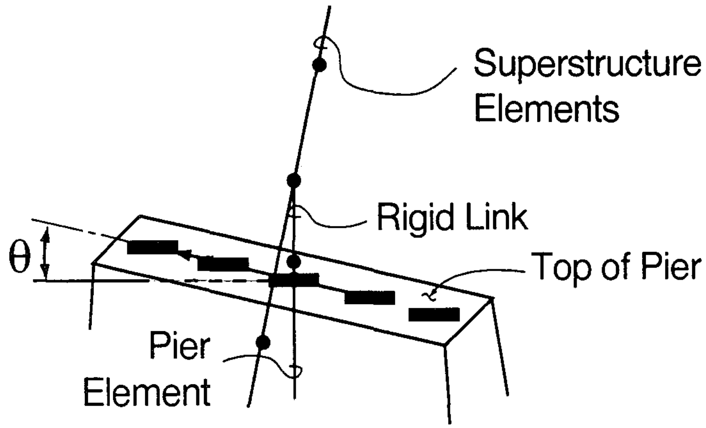


Plan View

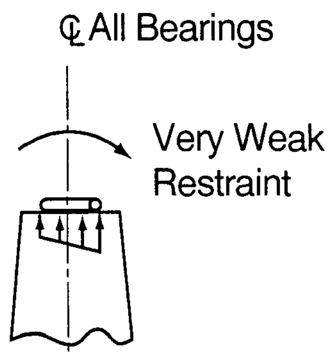
Section 1

- Selection of Seismic Model Releases Is Important

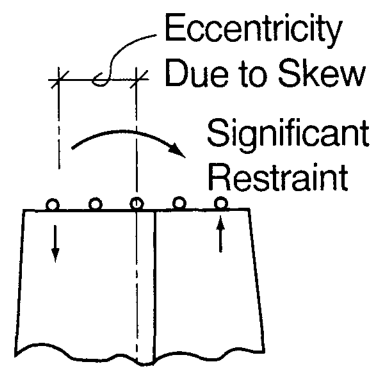
Release Directions for Bearings



Rotational Release for Pin Bearings



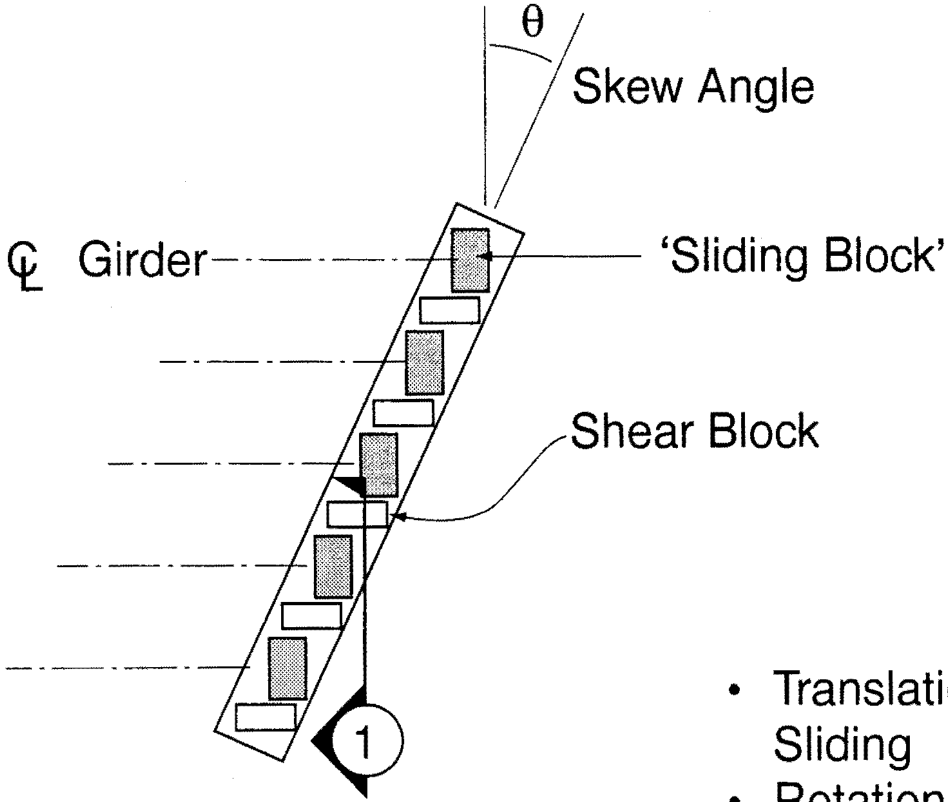
Looking Along Weak Axis of Pier



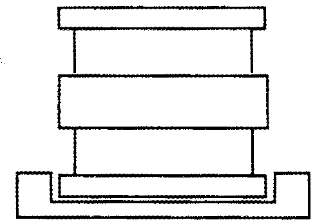
Elevation from Side of Bridge

Use Rotational Release About Weak Axis of Pier

Sliding Bearing Orientation



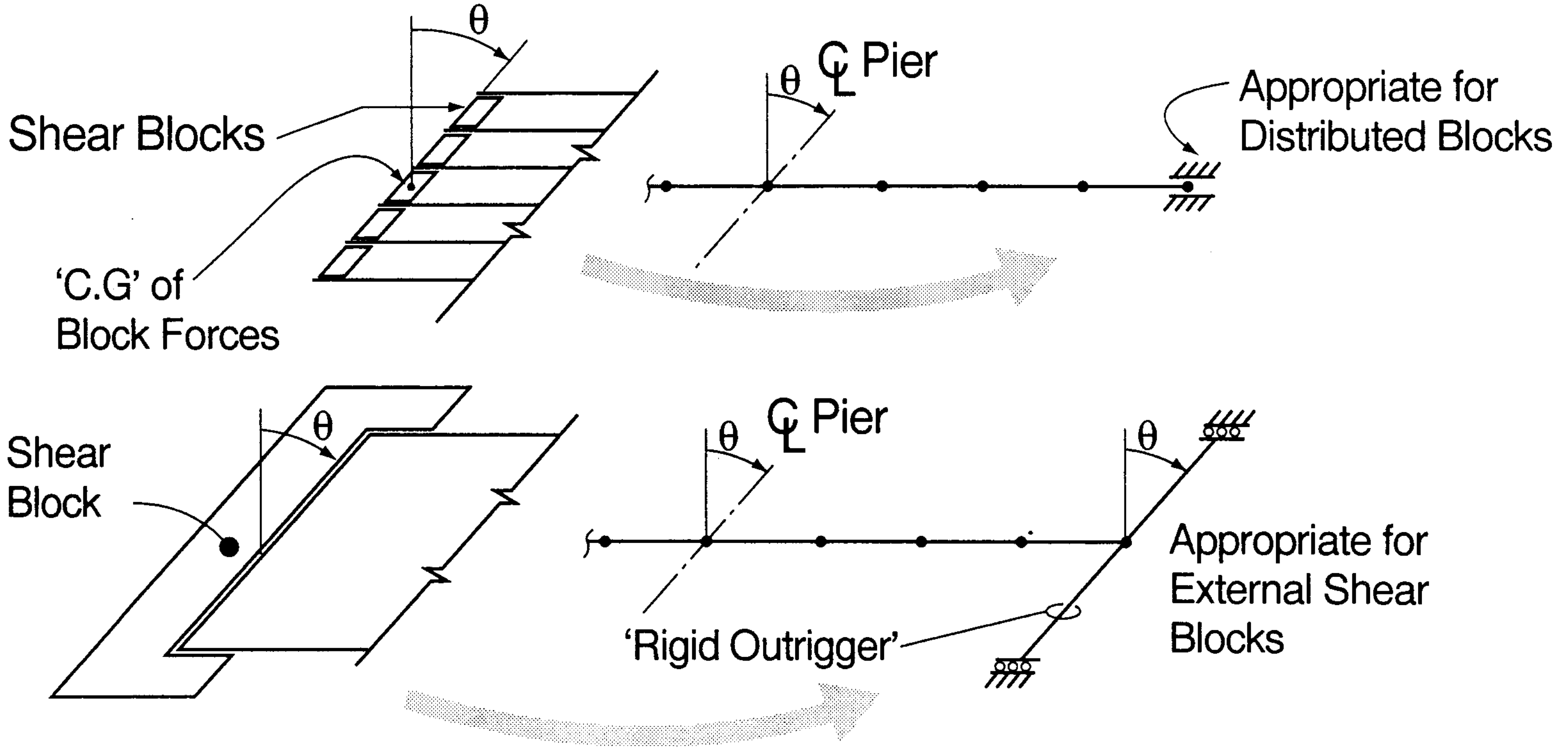
Plan View



Section 1

- Translational Release in Direction of Sliding
- Rotational Release About Weak Axis of Pier

Modeling Considerations for Shear Blocks

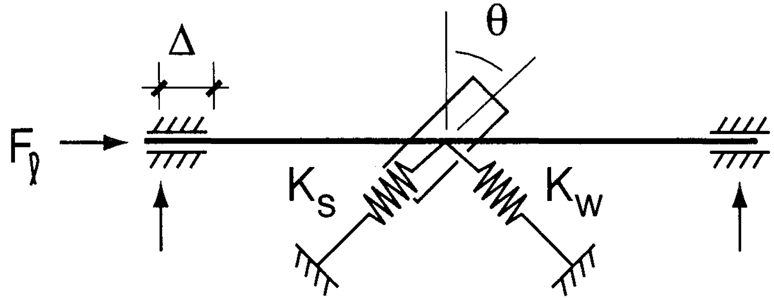


(Transverse Force Acts Only on One Side at a Time)

Consider Using Single-Mode Static Analysis for Severe Skew?

Stiffness Considerations (1 of 3)

Consider a Two-Span **Rigid** Deck System as Shown



Plan View

- For a Given Longitudinal Displacement, the Transverse Forces Developed by K_S and K_W Are Not Equal

∴ Transverse Reactions Are Required

Stiffness Considerations (2 of 3)

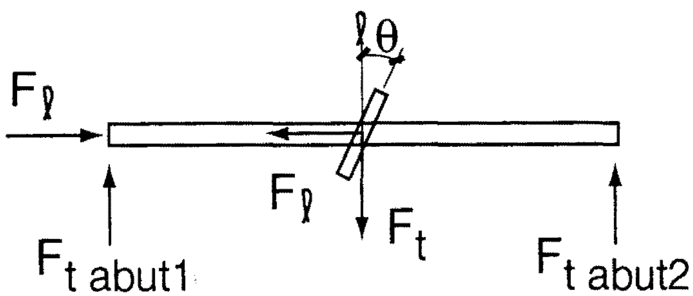
- **It Can Be Shown That**

$$K_\ell = K_s \sin^2\theta + K_w \cos^2\theta$$

Structure Stiffness in Longitudinal Direction

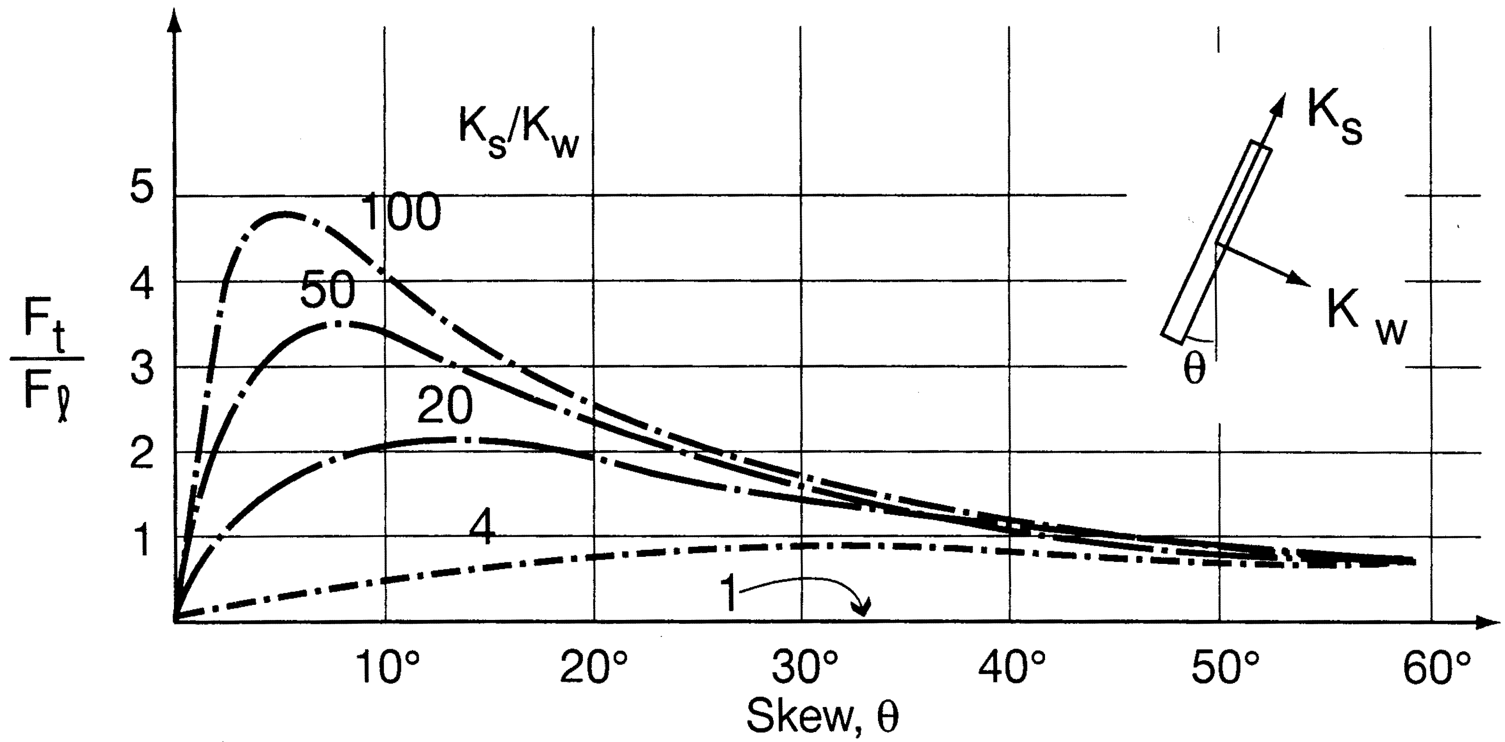
$$\frac{F_t}{F_\ell} = \frac{(K_s - K_w)\sin\theta\cos\theta}{(K_s \sin^2\theta + K_w \cos^2\theta)}$$

Ratio of Transverse Force to Longitudinal Force for a Given Displacement



Plan View

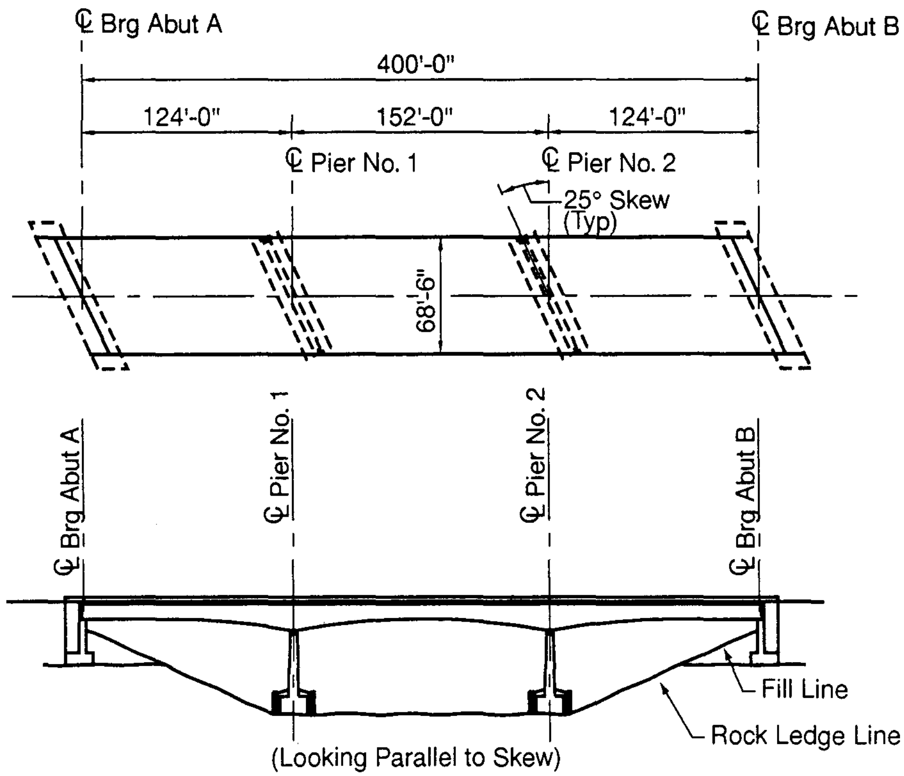
Stiffness Considerations (3 of 3)



- For Infinitely Stiff Superstructures, Large Transverse Forces May Develop!

Example / Effects of Skew (1 of 6)

- Consider Practice Problem No. 2 with 25° Skew

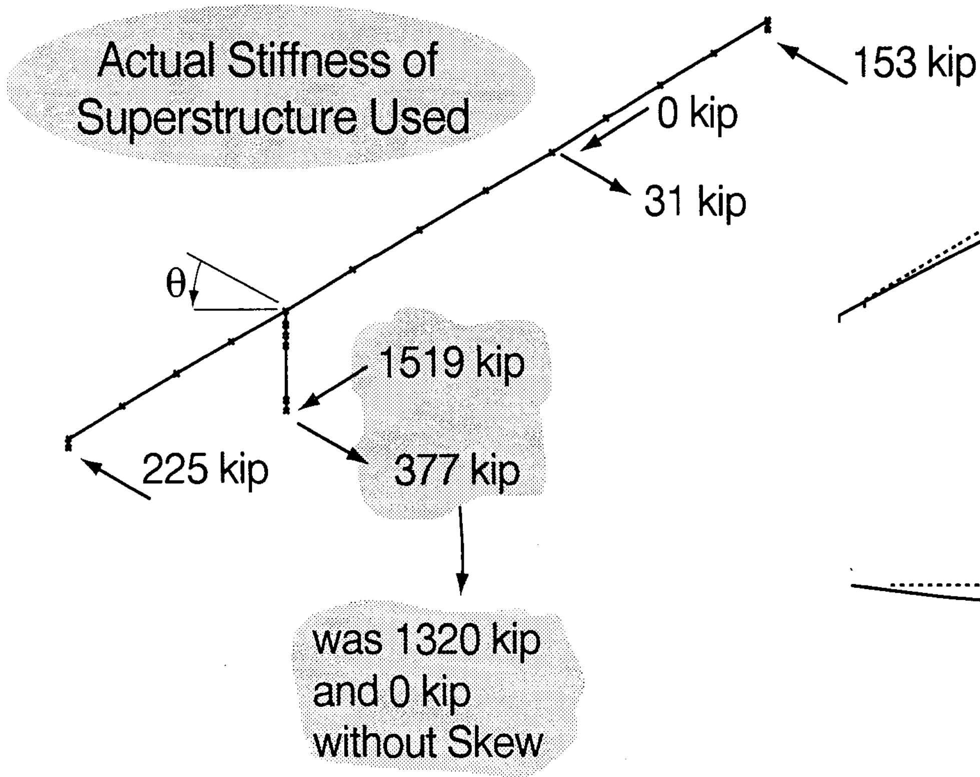


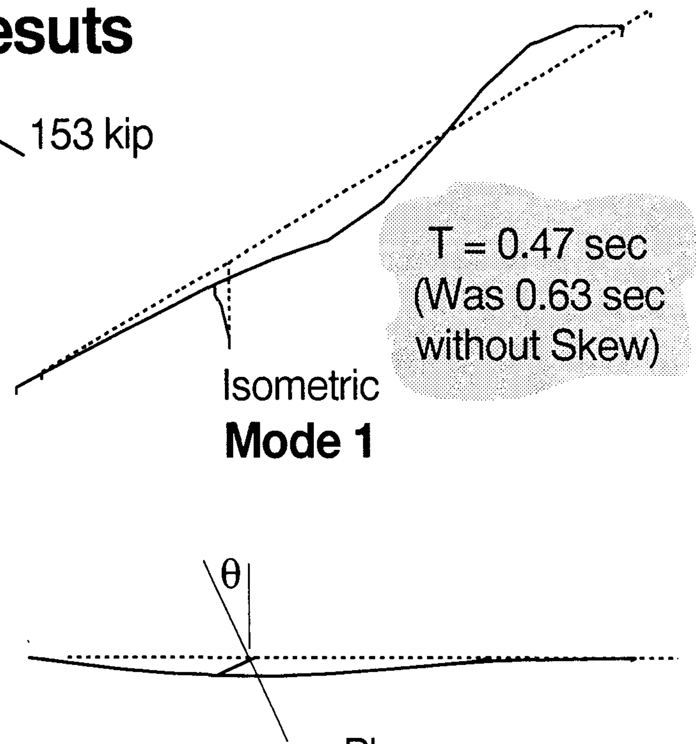
Example / Effects of Skew (2 of 6)

Determine the Longitudinal Base Shear and Transverse Restraint Forces by Frame Analysis and by Hand for Longitudinal Earthquake

Example / Effects of Skew (3 of 6)

Computer Frame Analysis Resuts





Isometric
Mode 1

Plan
Mode 1

Example / Effects of Skew (4 of 6)

Hand Analysis (Assume Rigid Superstructure)

Recall Pier Stiffness, $K_{weak} = 27150$ kip/ft

Seismic Weight, $W = 6041$ kip

Strong Direction Pier Stiffness → Approximate Using:

Width = 60 ft, Thk = 5 ft

H = 36 ft

E = 519,000 ksf   G = 220,000 ksf

Example / Effects of Skew (5 of 6)

$K_{strong} = 1{,}140{,}000$ kip/ft $\qquad \dfrac{K_s}{K_w} = \dfrac{1140000}{27150} = 42$

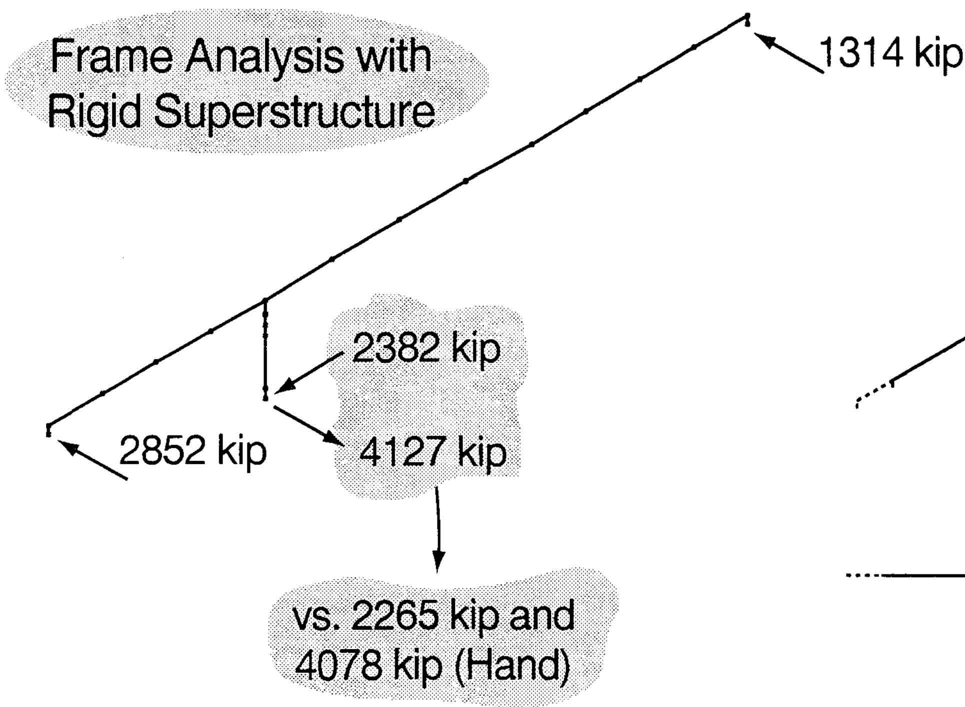
Using Plot, $\theta = 25°$ $\dfrac{F_t}{F_\ell} = 1.8$

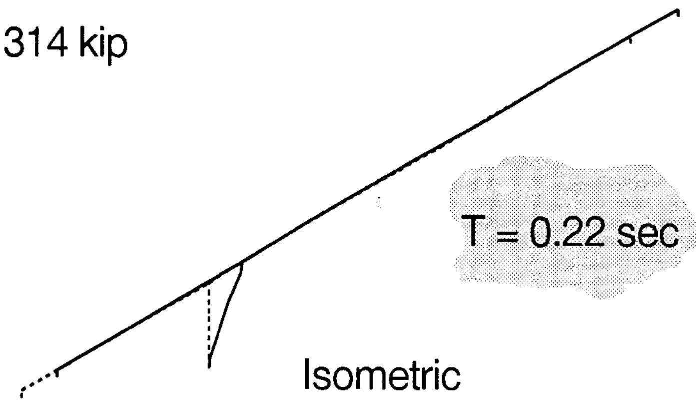
$$K_{long} = K_s \sin^2\theta + K_w \cos^2\theta = \underbrace{205{,}200 + 22{,}300}_{9.2/1} = 227{,}500 \text{ kip/ft}$$

$T = 0.18$ sec $\quad C_s = 0.375 \quad V_\ell = 2265$ kip

$F_t = 1.8\ (2265) = 4078$ kip (vs. 377 from Frame Analysis)

Example / Effects of Skew (6 of 6)



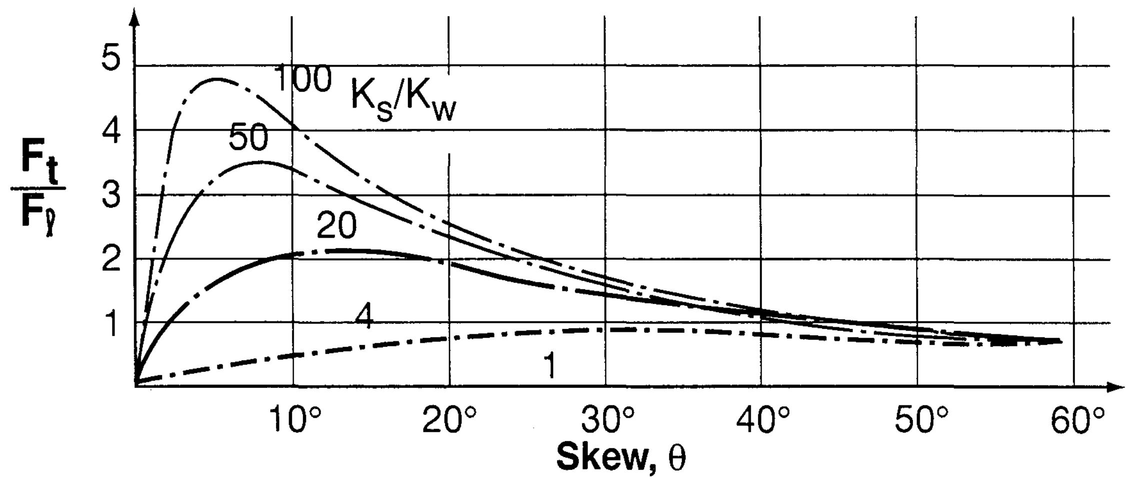


Isometric
Mode 1

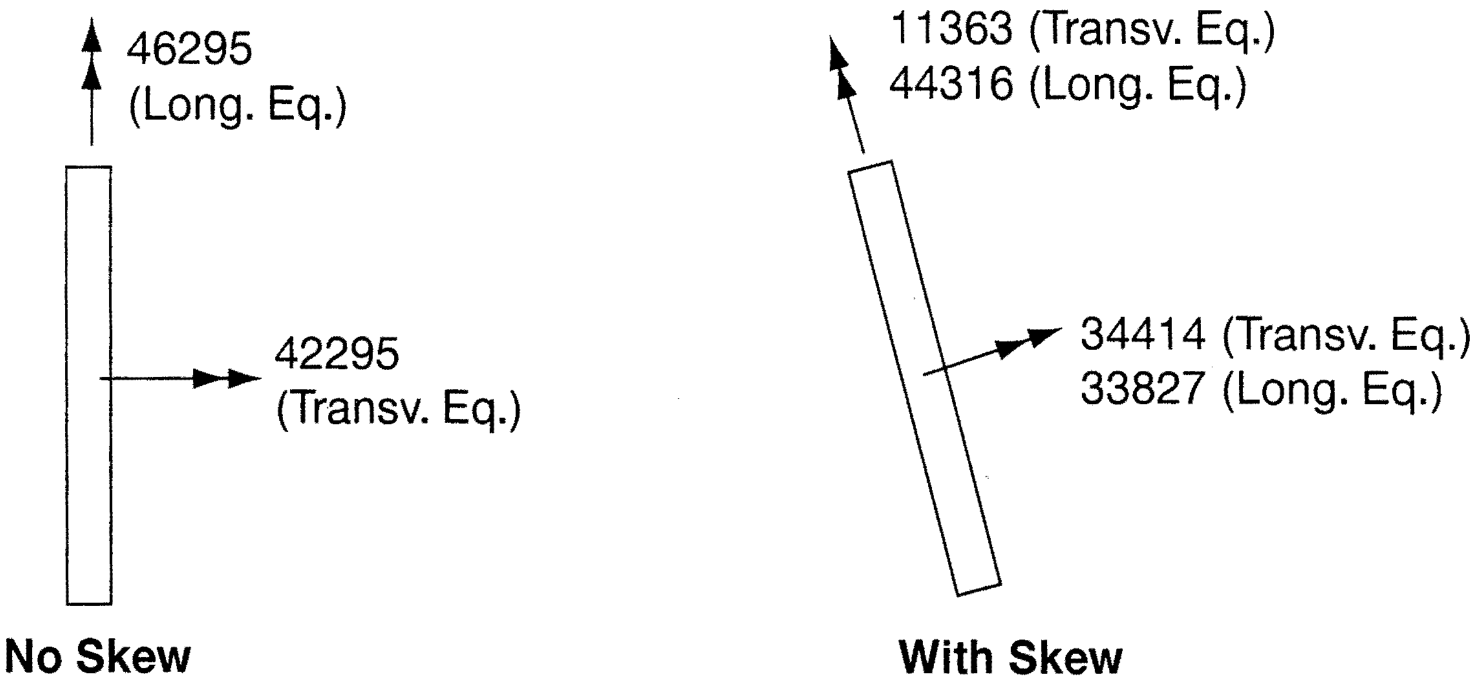
Plan
Mode 1

Relative Stiffnesses

K_s/K_w = 1 ...... Round Columns Fixed Top and Bottom
K_s/K_w = 4 ...... Columns Fixed Top in Strong Direction and Free Top in Weak Direction
K_s/K_w = 20 ..... Rectangular Columns or Walls
K_s/K_w = 50+ ... Walls, But Superstructure Not Rigid, Relative to Stiff Walls, Need Frame Analysis

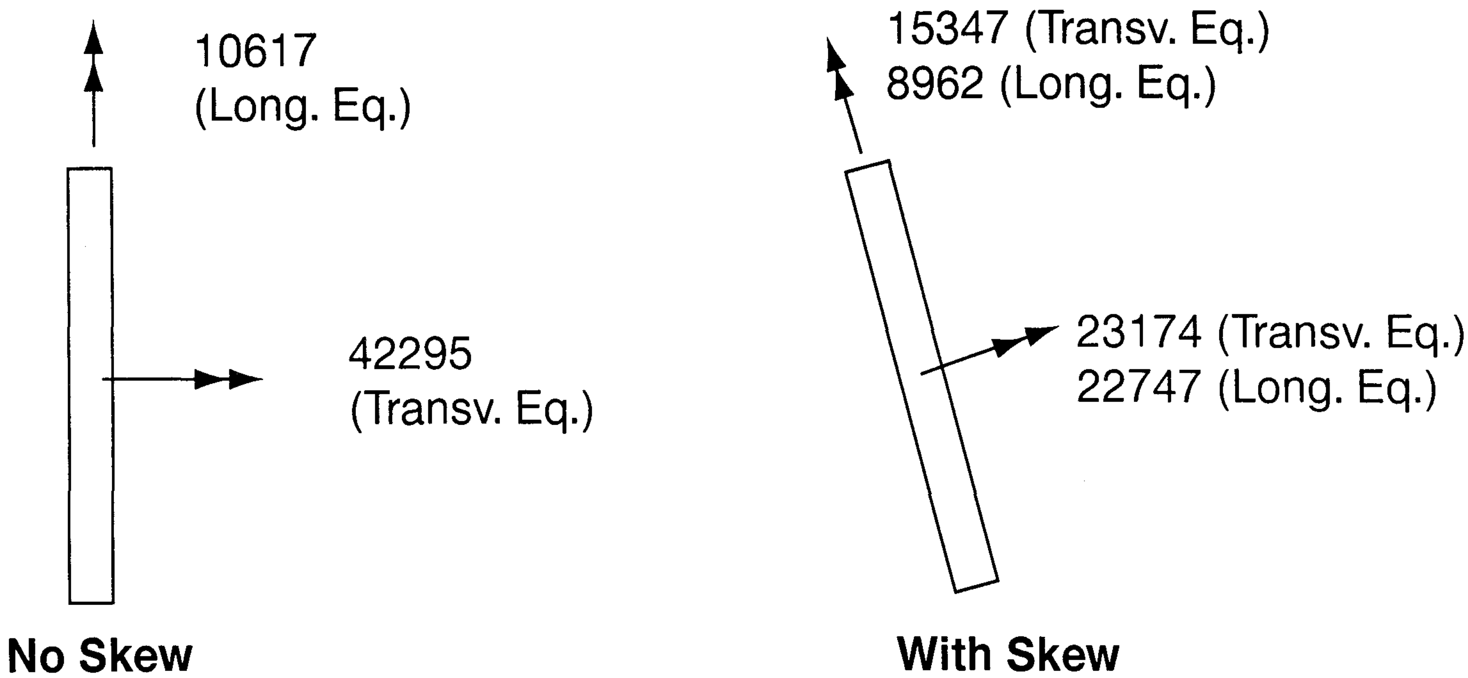


Example Pier No. 1 – Moments of Base of Wall



Moments in kip ft

Example Pier No. 2 – Moments of Base of Wall



Moments in kip ft

Example / Effects of Skew

Summary

- Coupling of Longitudinal and Transverse Forces Can Be Significant
- Coupling Very Sensitive to Relative Stiffness of Superstructure and Piers

Implications

- For Stiff Superstructure / Flexible Pier Bridges, Shear Block Forces Can Be Quite High
- Failure of Shear Blocks Will Induce Torsional Response (Worsens: Seating and Outer Column / Pier Response)

Minimizing Effects of Skew

- Elastomeric Bearing Pads, Which Can Have Omnidirectional Flexibility for Both Translation and Rotation, Can Help Minimize Effects of Skew

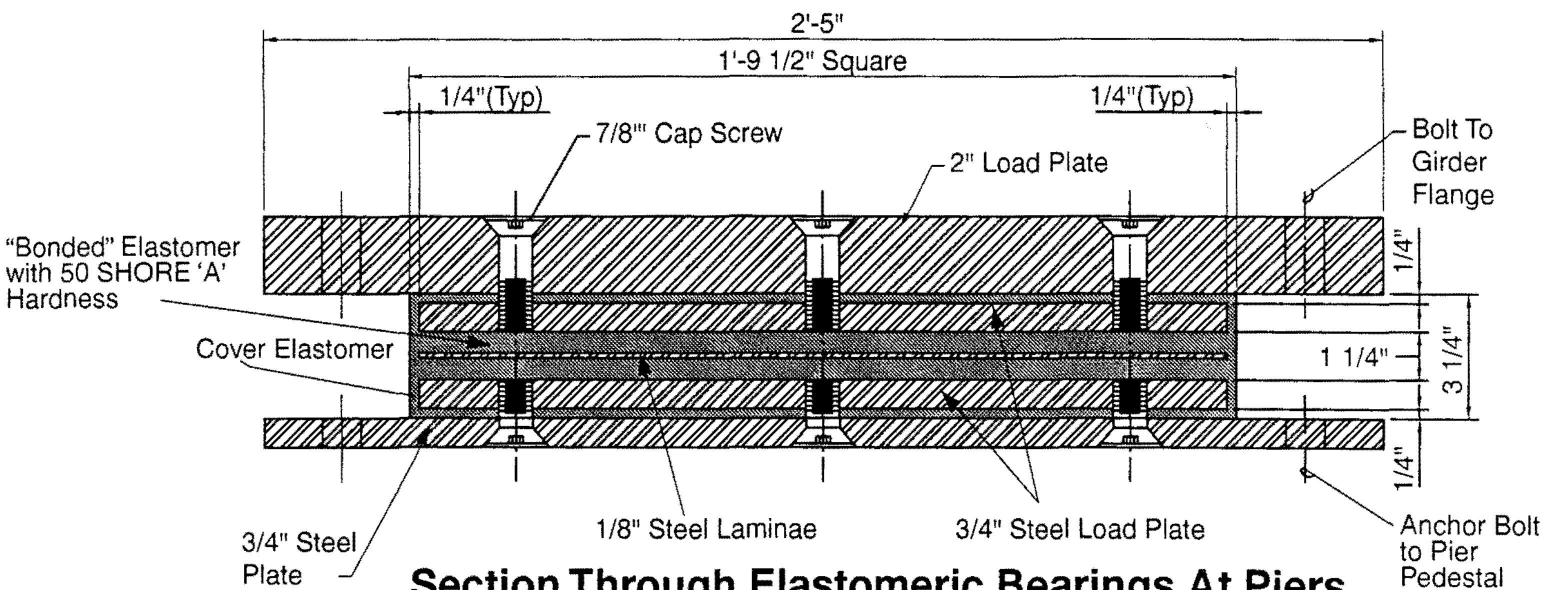
- For Example, See Design Example No. 2

Session 4
Elastomeric Bearing and Modeling Design

- **Concepts and Configuration**
- **Stiffness Calculations**
- **Limiting Strain**
- **Details**

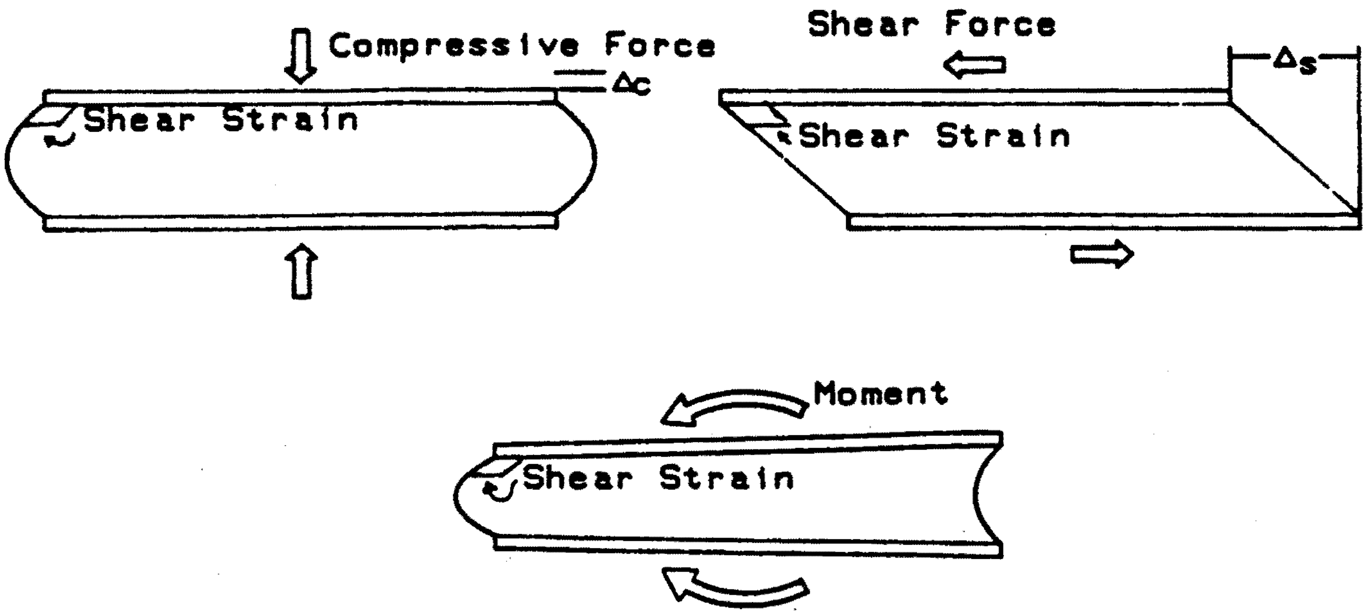
(These Are Not Seismic Isolation Bearings)

Bearing Configuration



Section Through Elastomeric Bearings At Piers

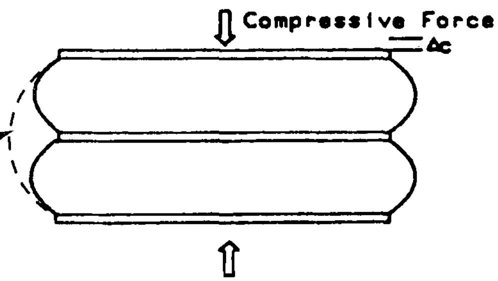
Conceptional Behavior

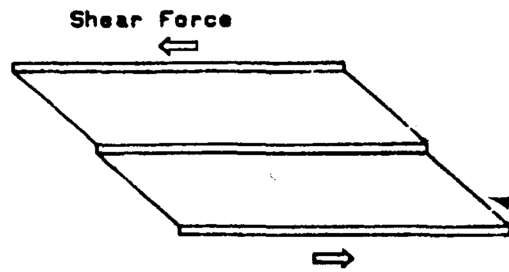


- All Loadings Induce Shear Strains

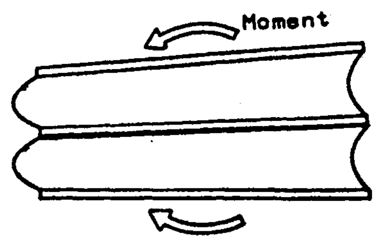
Roeder and Stanton (1990)

Behavior with Stiffeners





Stiffener Restrains Bulging of Elastomer

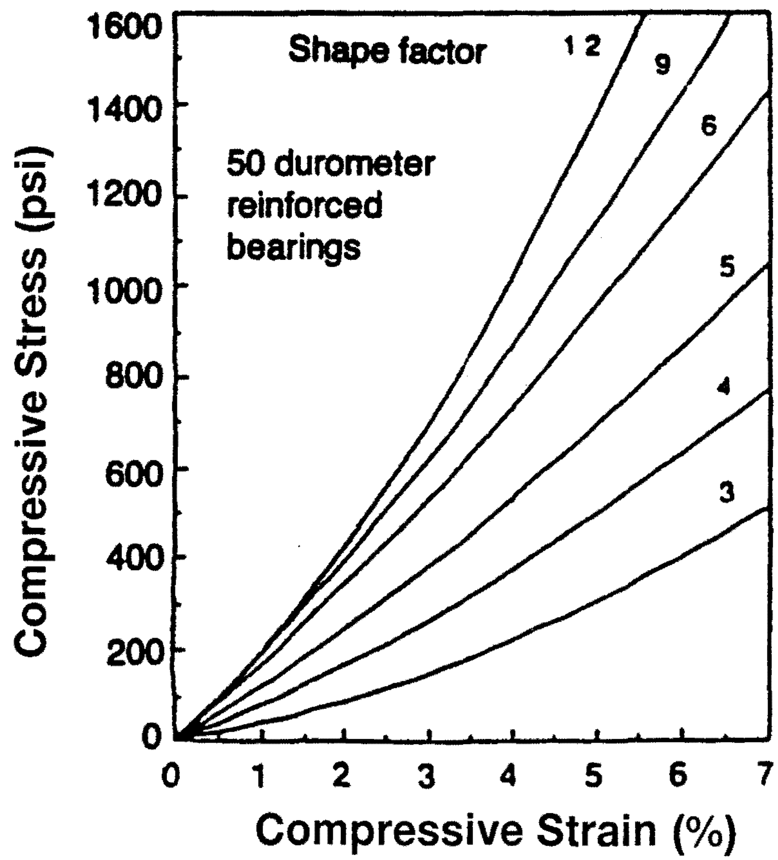


Shearing Deformations Take Place in Elastomer Only

Properties of Elastomer

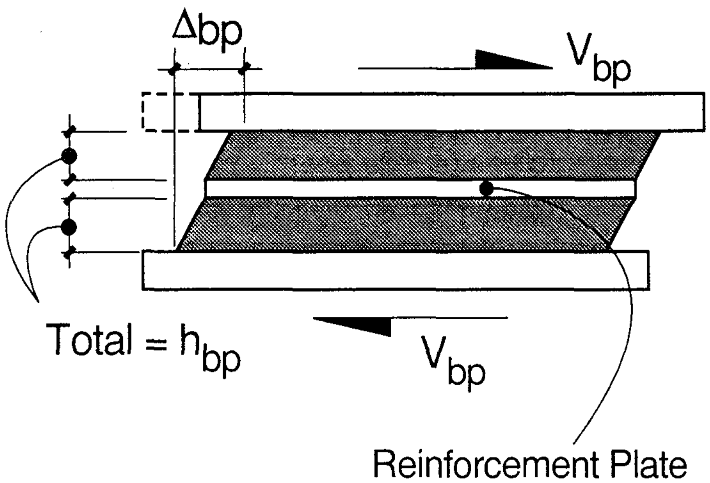
Hardness (Shore 'A')	Elastomer Shear* Modulus, G (psi)
50	95 - 130
60	130 -200
70	200 - 300

* Coordinate with Supplier



AASHTO (1995) (Division I)

Stiffness Calculation for Lateral Loads



$$K_h = \frac{V_{bp}}{\Delta_{bp}} = \frac{GA}{h_{bp}}$$

A = Area of Bonded Elastomer

h_{bp} = Total Height of Elastomer

(Do Not Include Reinforcement Plates)

Stiffness Calculation for Vertical Loads

- Shape of Bearing Affects Stiffness

$$\text{Shape Factor, } S = \frac{\text{Plan Area}}{\text{Perimeter Area Free to Bulge}} = \frac{LW}{2h_{ri}\,(L + W)}$$

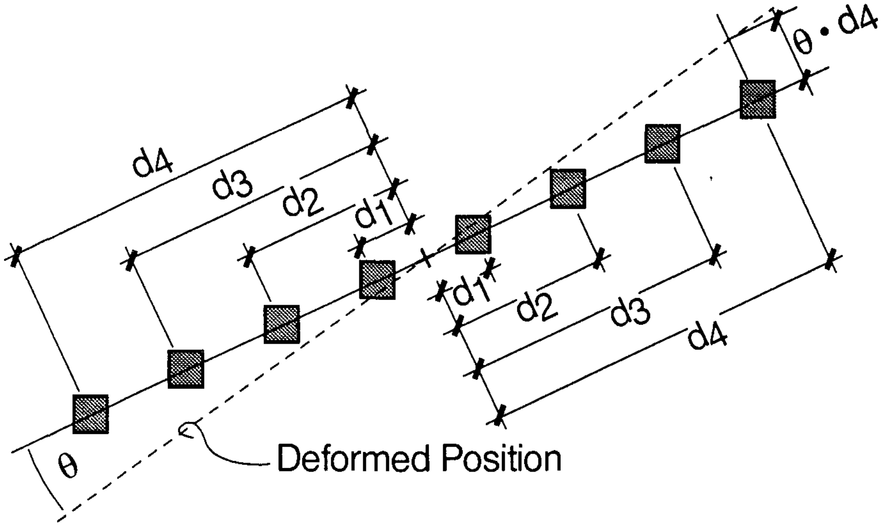
L = Length
W = Width

h_{ri} = Height of Layer

- Based on Compressive Stress and Shape Factor, Calculate Strain and Then Displacement

- Find Stiffness from Compression Force and Displacement

Rotational Stiffness of Group



Plan View
Bearing Pads on Skew Pier

$$K_{rot} = \frac{M}{\theta} = \sum_{i=1}^{n} K_{brg_i} d_i^2$$

K_{brg_i} = Individual Bearing Translational Stiffness

d_i = Distance from Centroid to Bearing i

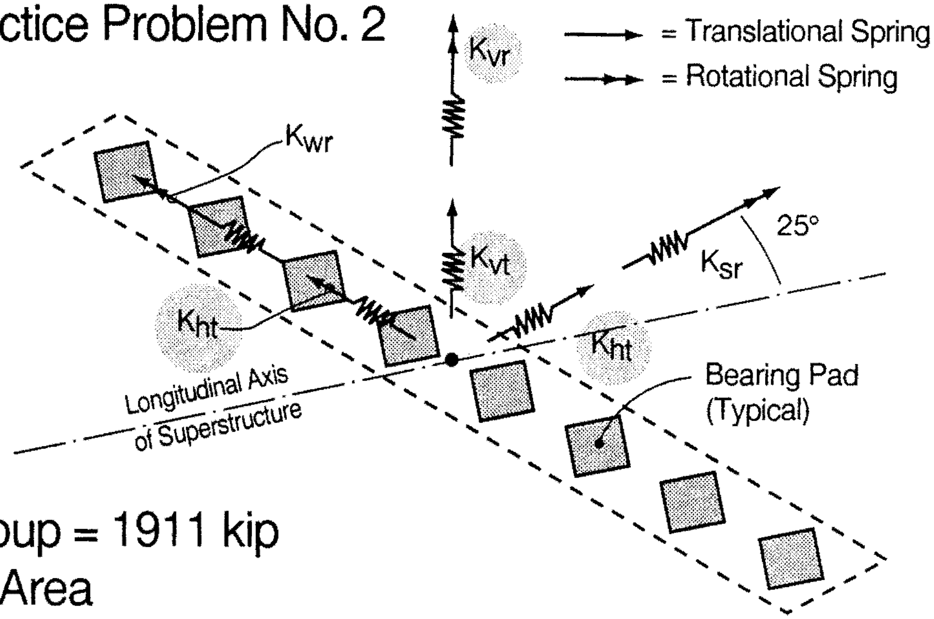
- Vertical Rotational Stiffness Similar

Example / Elastomeric Bearing Stiffness (1 of 5)

- Consider Bearing Shown at Beginning of Section and 25° Skew Bridge of Practice Problem No. 2

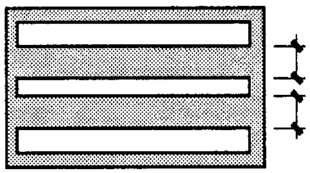
- Calculate Stiffness Shown in Shaded Bubbles

- Use G = 115 psi
  Weight on Bearing Group = 1911 kip
  21 in. x 21 in. Bonded Area



Configuration at Pier

Example / Elastomeric Bearing Stiffness (2 of 5)



$h_{bp} = 1.125$ in.

- One Pad: $$k_{ht} = \frac{GA_{bp}}{h_{bp}} = \frac{115(21)^2\ 12}{1.125\ (1000)} = 541 \text{ kip/ft}$$
- Eight Pads: $K_{ht} = 8\ (541) = 4328$ kip/ft
- Note that Stiffness Is the Same in All Directions

Example / Elastomeric Bearing Stiffness (3 of 5)

- Stress on Individual Bearings

$$\sigma = \frac{1911\ (1000)}{8(21)^2} = 542 \text{ psi}$$

- Shape Factor

$$h_{ri} = \frac{1.125}{2} = 0.563 \text{ in.} \qquad S = \frac{LW}{2h_{ri}\ (L+W)}$$

$$S = \frac{(21)^2}{2(0.563)(21+21)} = 9.3$$

- From AASHTO Plot (50 Durometer)

Compressive Strain $\varepsilon_c = 0.025$ (Use Manufacturer's Data if Available)

Example / Elastomeric Bearing Stiffness (4 of 5)

- **One Pad**

$$k_{vt} = \frac{AE}{h_{bp}} \bullet \frac{A\,\sigma/\varepsilon}{h_{bp}} = \frac{(21)^2\left(\frac{0.542}{0.025}\right)(12)}{(1.125)}$$

$$k_{vt} = 102{,}000 \text{ kip/ft}$$

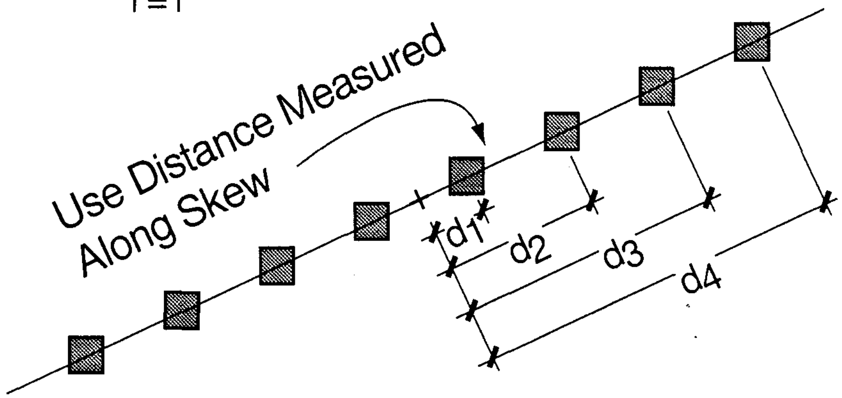
- **Eight Pads**

$$K_{vt} = 8(102{,}000) = 816{,}000 \text{ kip/ft}$$

Example / Rotational Stiffness About Vertical Axis (5 of 5)

$$K_{rv} = \sum_{i=1}^{8} k_{ht} d_i^2 = 2\,k_{ht} \sum_{i=1}^{4} d_i^2$$

$d_1 = 4.5$ ft
$d_2 = 13.5$ ft
$d_3 = 22.5$ ft
$d_4 = 31.5$ ft



Plan View

$$K_{rv} = 2(541)\left[4.5^2 + 13.5^2 + 22.5^2 + 31.5^2\right] = 1{,}841{,}000\ \frac{\text{kip ft}}{\text{rad}}$$

Assessing Seismic Performance

- **Conventional** (Division I) → **Limit Service Shear Displacement** (To 1/2 Elastomer Height)
- **Seismic Loadings** → **Assess Against Ultimate Resistance** (Not Service Allowable)
- **Suggest AASHTO's Guide Specification for Seismic Isolation Design**
  (Use Article 14.6, Seismic Load Combinations, Even Though We Are Considering Only Conventional Elastomeric Bearings in This Section)

Assessing Seismic Performance of Conventional Elastomeric Bearings

Limit Strains to: $0.75\,\varepsilon_u > \varepsilon_{sc} + \varepsilon_{eq} + \varepsilon_{sr}$

AASHTO Seismic Isolation Guide Specification / §14.6

$\varepsilon_u =$ Minimum Elongation-At-Break of Elastomer (From AASHTO or Preferably Supplier)

Example, Table 18.2.3.1 Division II
50 Durometer Neoprene $\varepsilon_u = 400\%$

$\varepsilon_{sc} =$ Shear Strain Due to Compression $= 65\,S\,\varepsilon_c$

Compressive Strain

$\varepsilon_{eq} =$ Shear Strain Due to Earthquake $= \Delta_{eq} / h_{elastomer}$

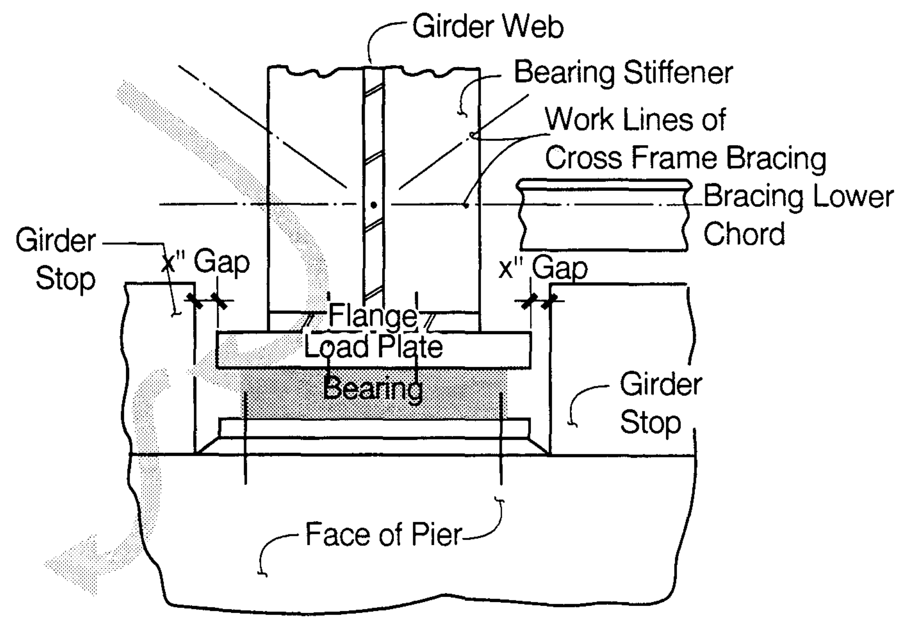
Load Direction Dimension

$\varepsilon_{sr} =$ Shear Strain Due to Rotation $= \dfrac{B^2\theta}{2h_{ri} \cdot h_{bp}}$

Fail-Safe Issues

- Consider an Additional Load Path in Case of Bearing Failure

- Engage Alternate Path After Bearing Deformation Occurs

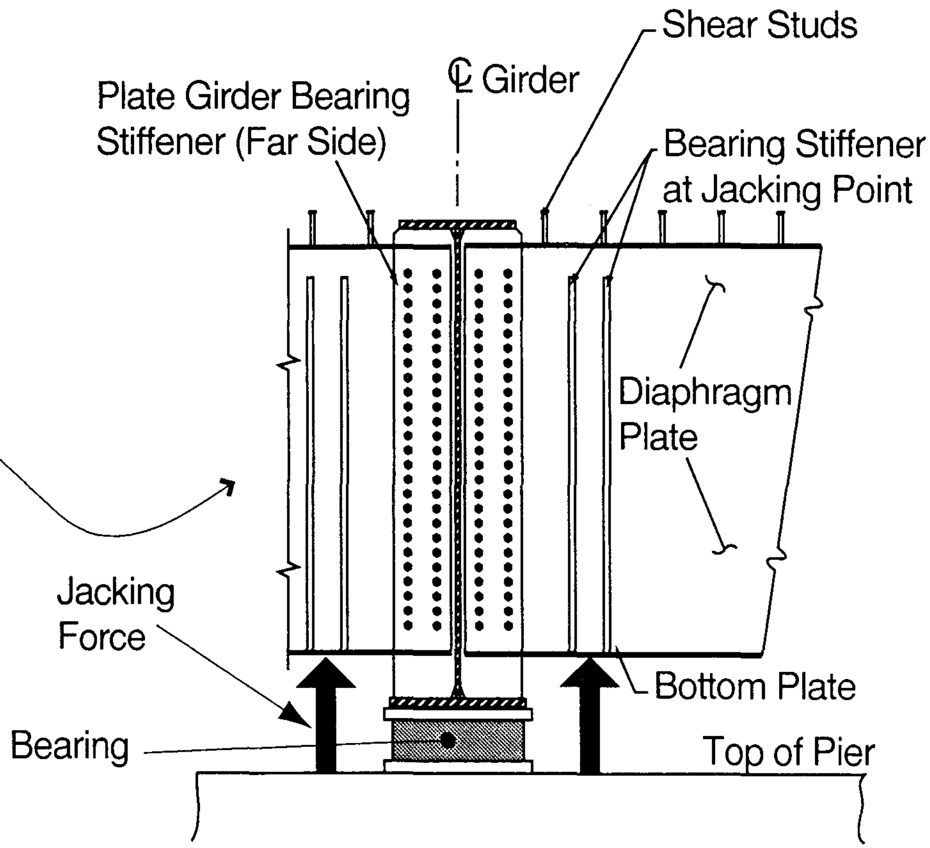


Consider Method of Bearing Replacement

- Lift Along Girder (Previous Sketch)

or

- Lift Along Solid Diaphragm-Type Cross Frame



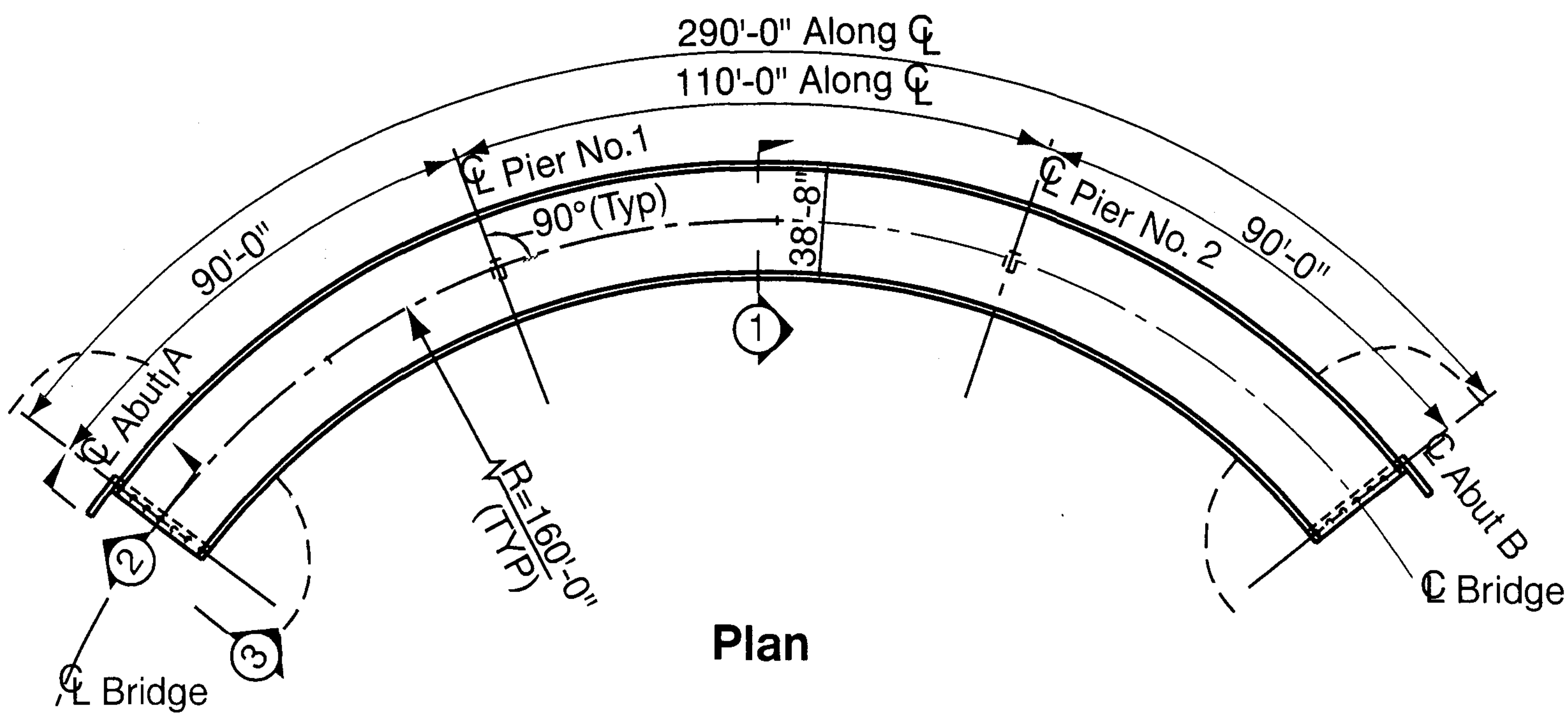
Interior Support Cross Frame Elevation

Session 5
Curved Box Girder Bridge Example

Session 5
- **Curved Structure Issues**
- **Piles**

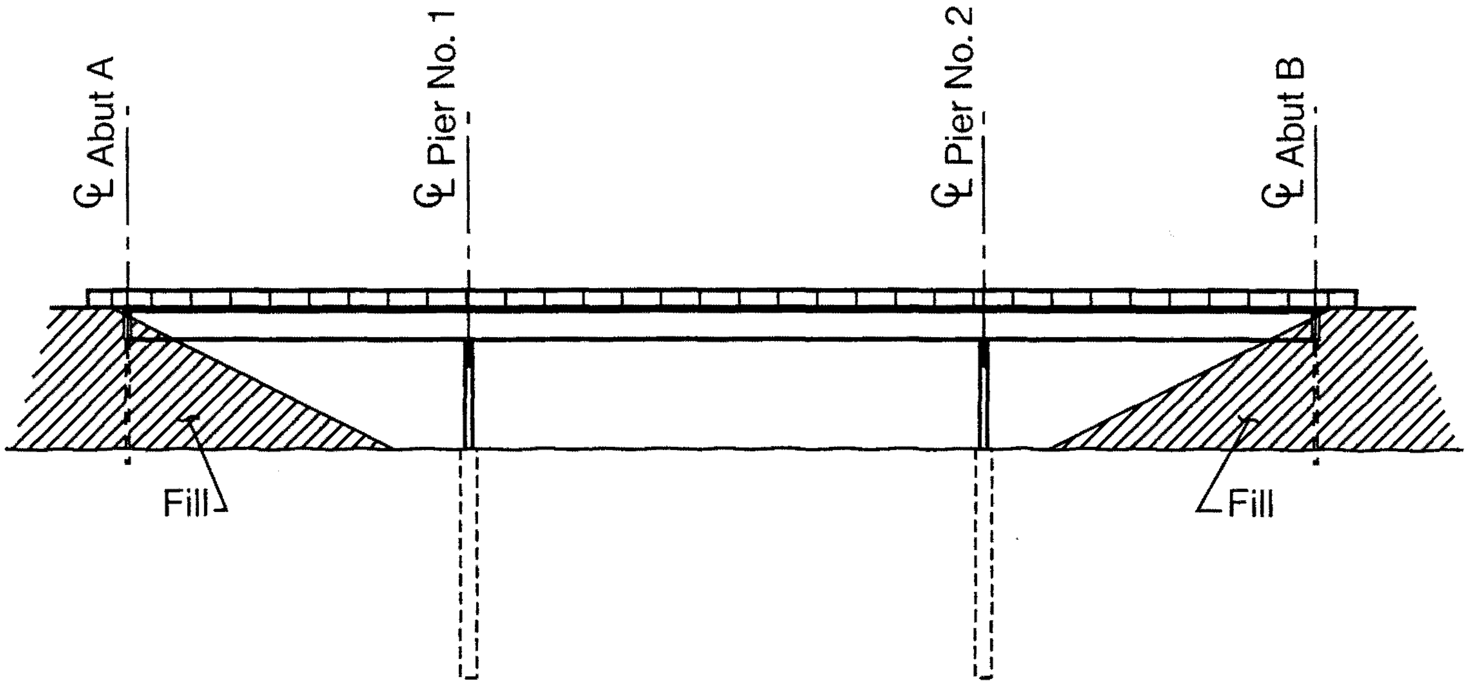
Session 6
- **Drilled Shafts**

Concrete Curved Box Girder Bridge / Plan



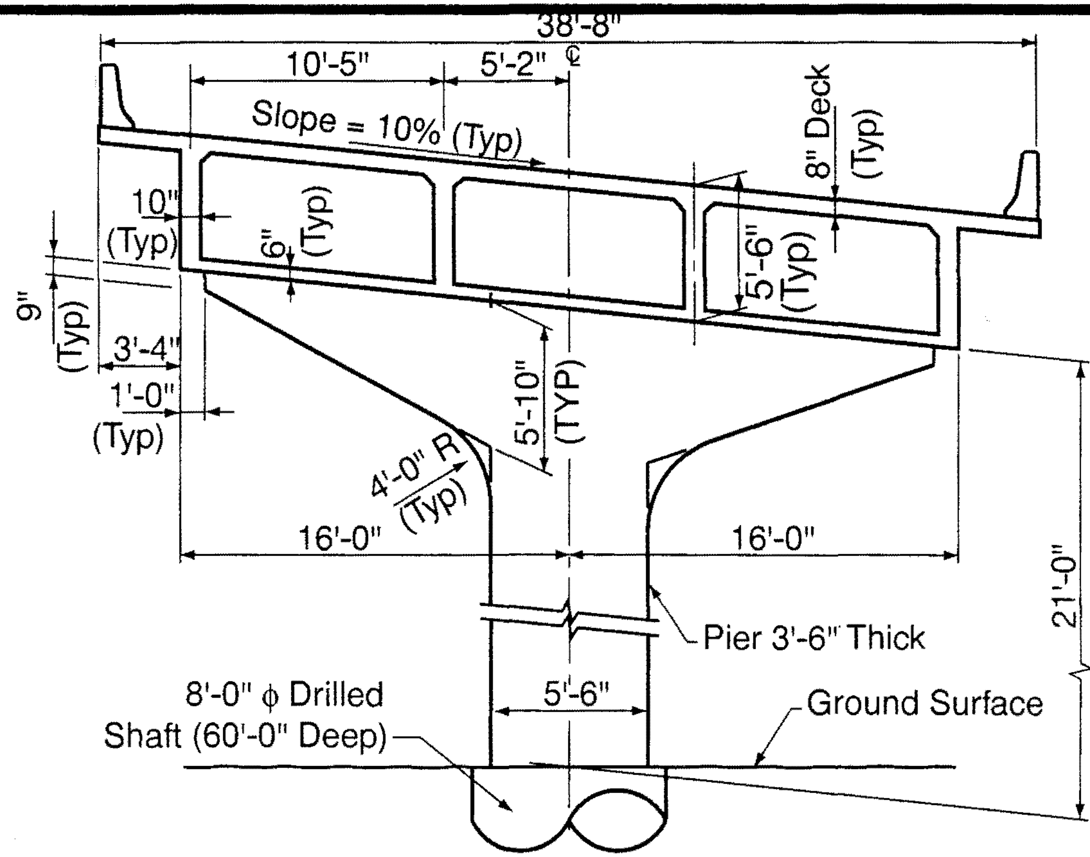
Plan

Concrete Curved Box Girder Bridge / Elevation



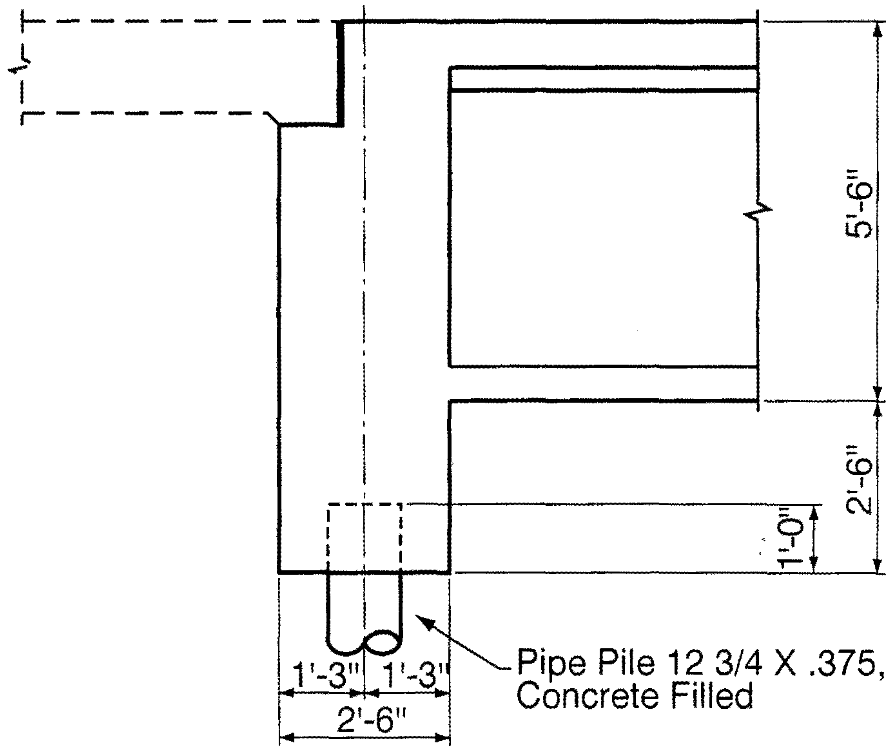
Developed Elevation

Concrete Curved Box Girder Bridge /Pier



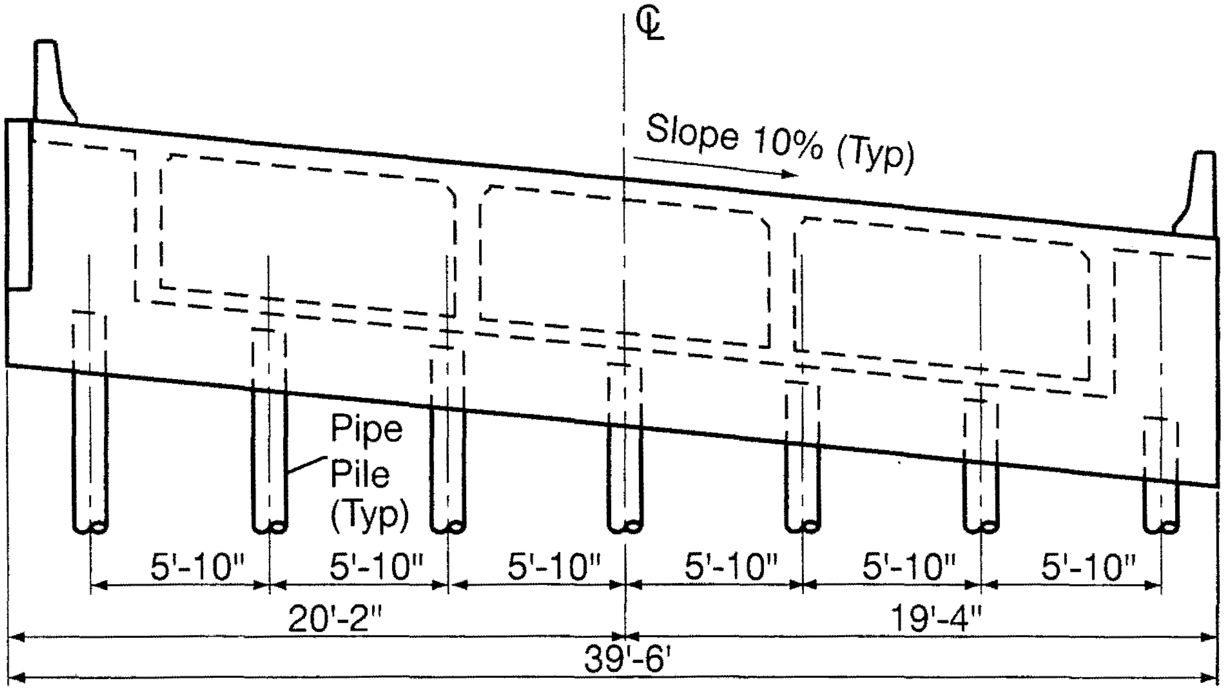
1 Section

Concrete Curved Box Girder Bridge / Abutment



2 Section

Concrete Curved Box Girder Bridge / Abutment



3 Elevation

Session 5

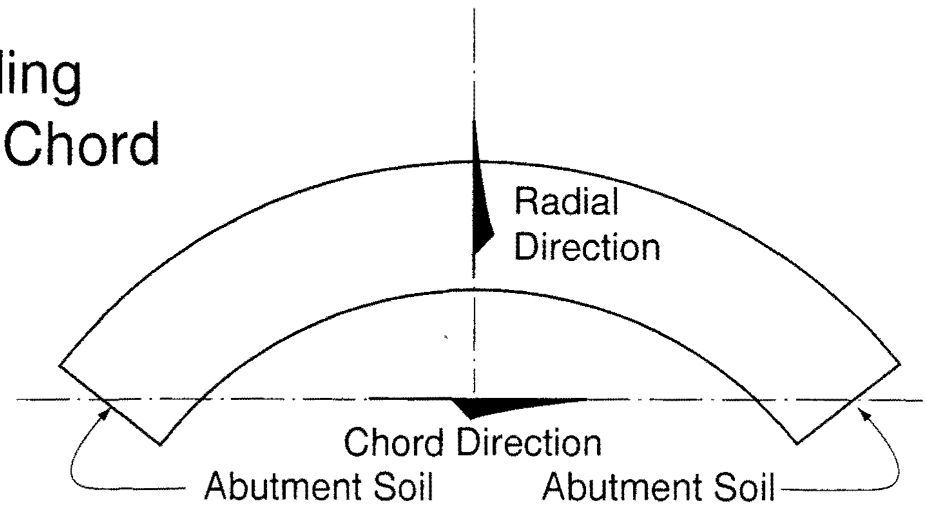
Curved Structure Issues

- Loading Directions
- Conceptual Behavior
- Bounding Response

AASHTO Loading Directions

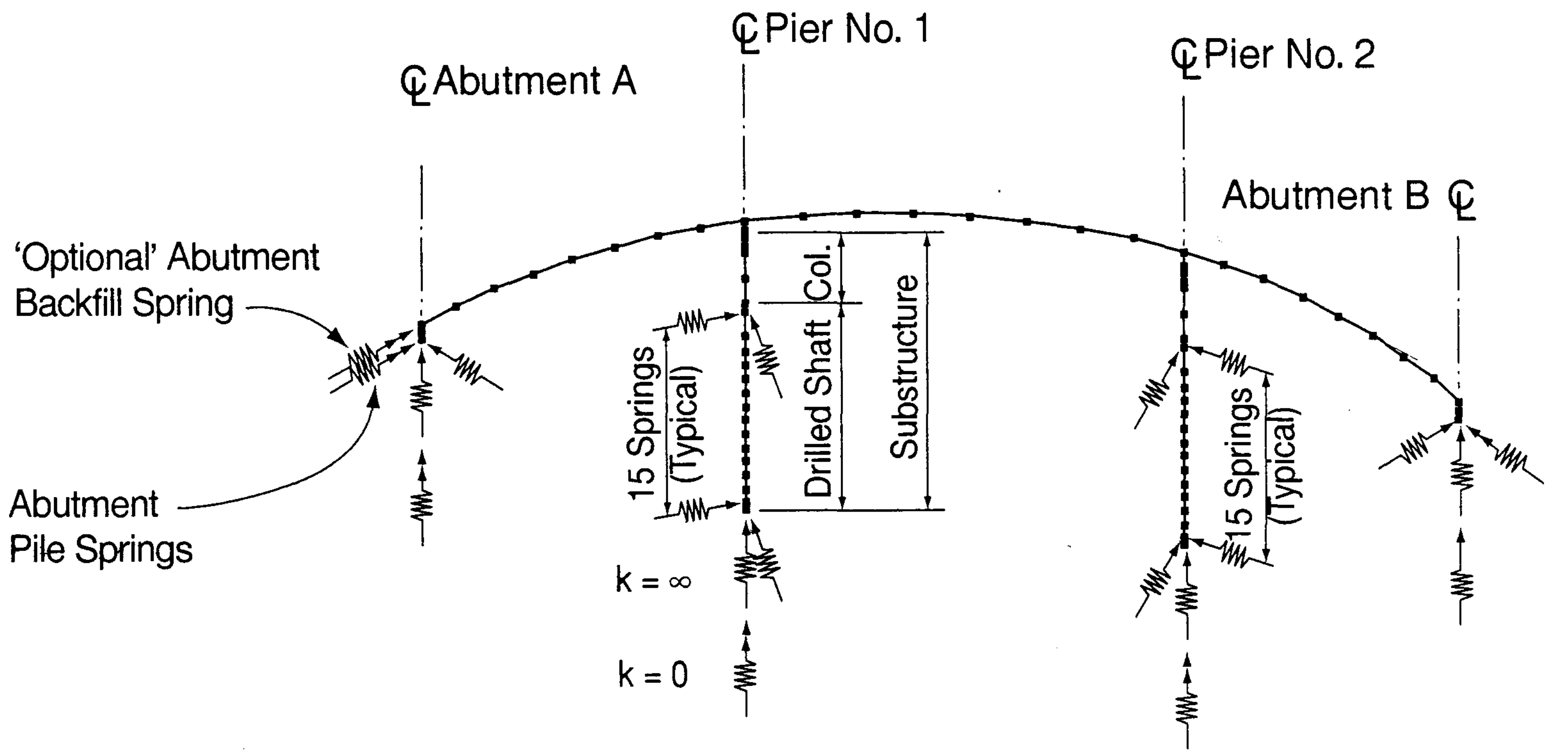
- If Modal Analysis Is Used (Required if 'Not Regular')

    1. Earthquake Loading Along Chord
    2. Earthquake Loading Perpendicular to Chord



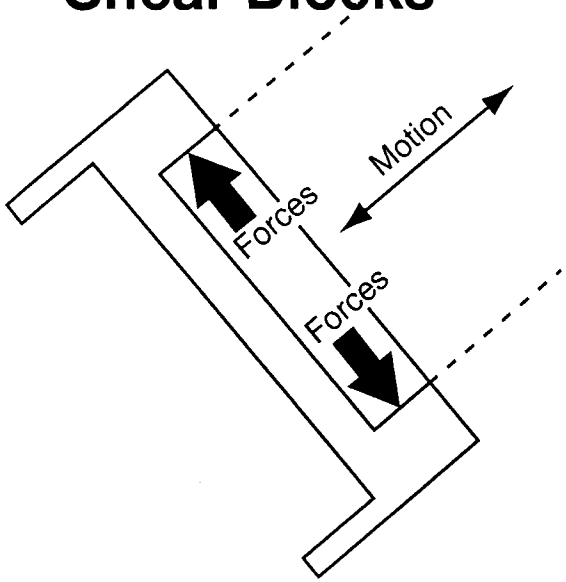
- Suggest the Same Loading Directions for Other Analysis Methods

Seismic Analysis Model / Example Bridge



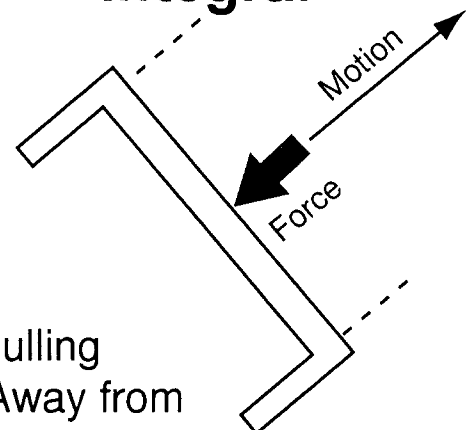
Effects of Abutment Restraint

Seat-Type with Shear Blocks



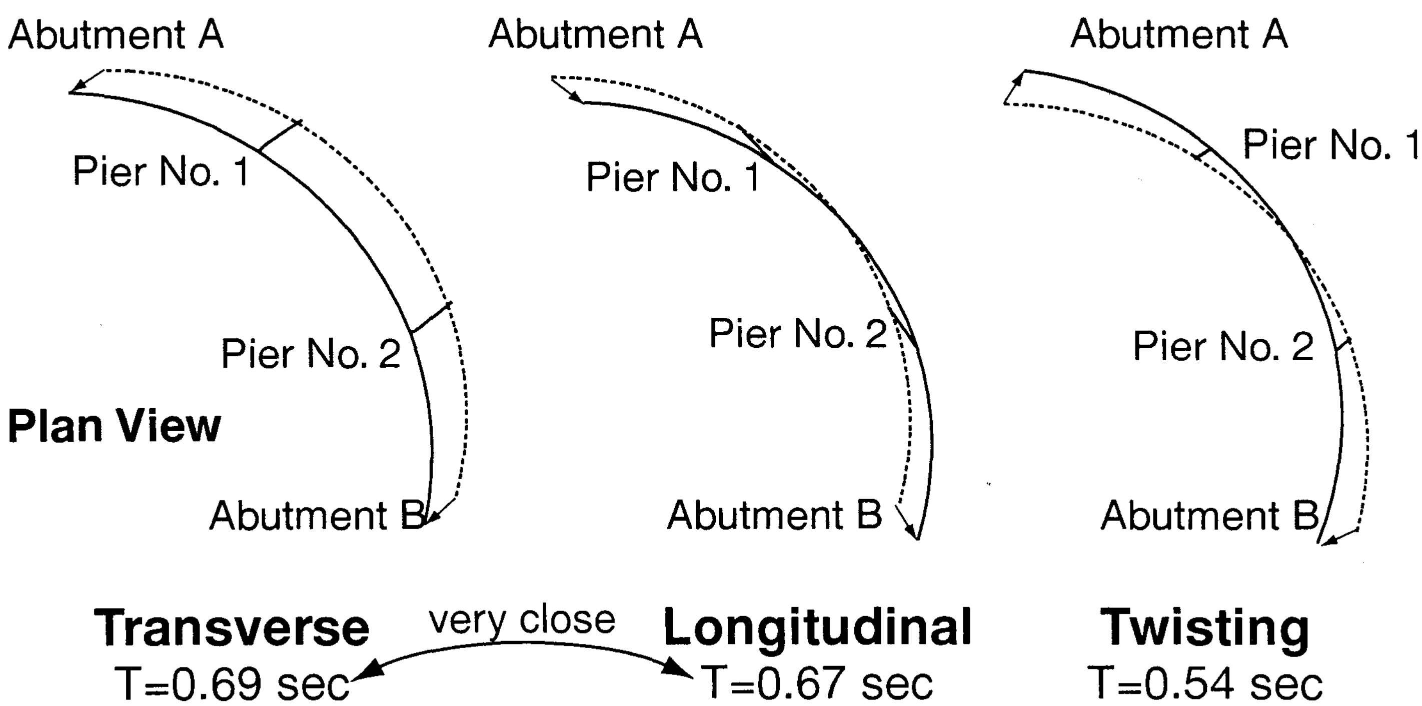
- Movement Will Heavily Load Shear Blocks

Integral

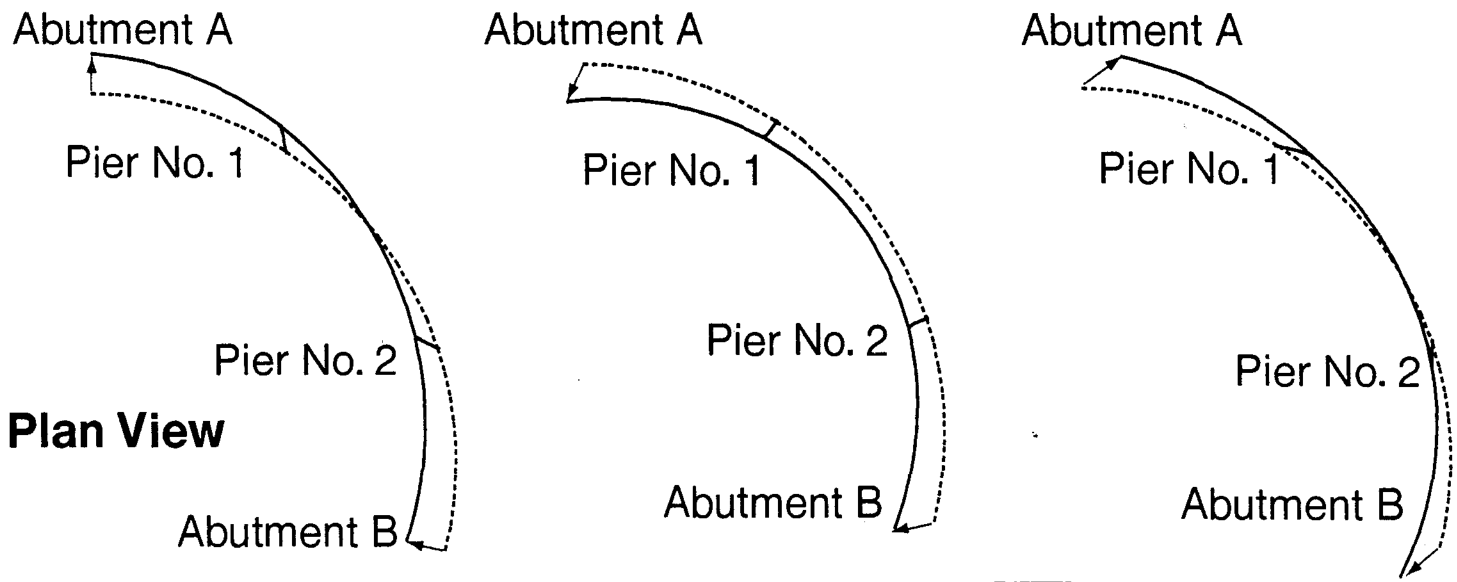


- Movement Pulling Diaphragm Away from Soil → Backfill Ineffective
- Movement Toward Soil Backfill Effective
- Use Bounding Analyses

Modal Behavior / No Backfill Considered



Modal Behavior / Including Backfill



Twisting
T=0.61 sec

Transverse
T=0.29 sec

Longitudinal
T=0.26 sec

Effects of Curve for Example Bridge

- Both Abutment Backfills Are Effective or Not Effective at the Same Time (Do Not Put 1/2 K to Each)

- No Backfill Case Controls
  - Piers / Drilled Shafts
  - Piles

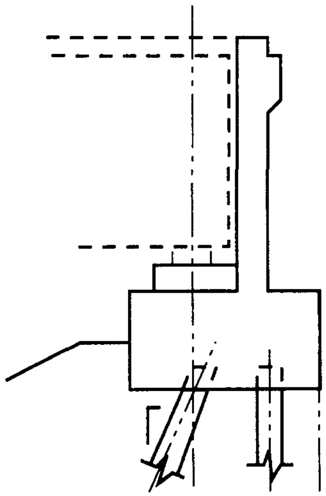
- Backfill Included Controls
  - End Diaphragm
  - Backfill Soil

- Torsional Stiffness of Superstructure Is More Influential in Forces Developed

Session 5
Piles

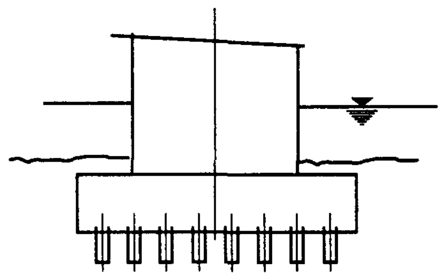
- Configuration and Behavior
- Including Flexibility in Analysis
- Coupling Effects
- Nonlinear Effects
- Multiple Pile Goups/Axial Stiffness
- Design and Detailing

Typical Configurations

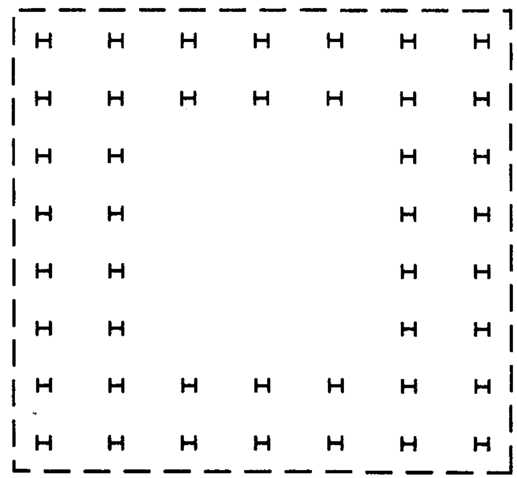


Piles Either Steel,
Concrete, or Timber

Abutment

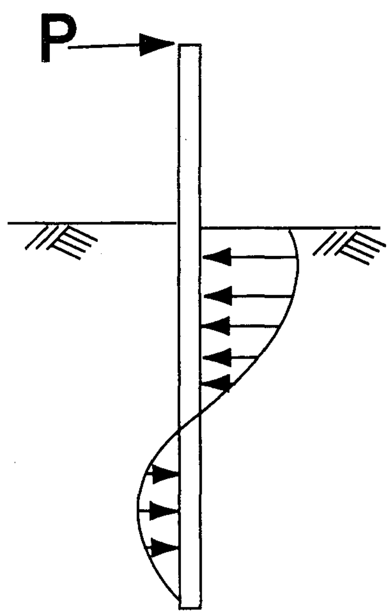


Pier Section

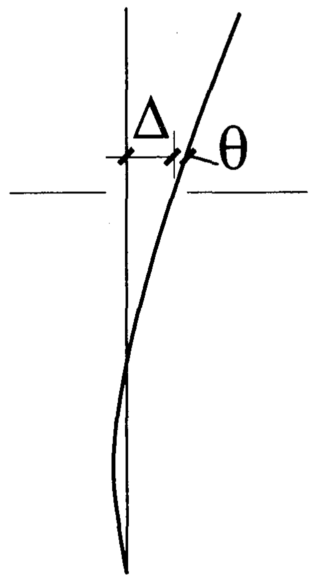


Pier Plan

Behavior Under Lateral Loading



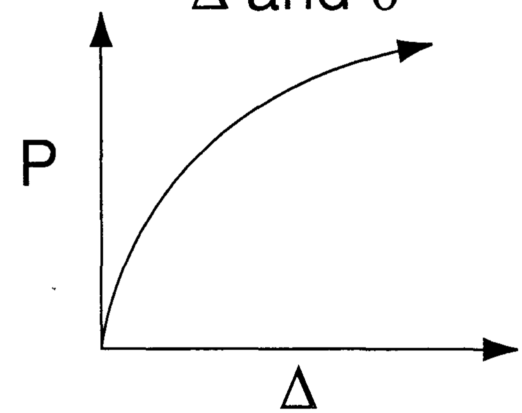
Forces on Pile



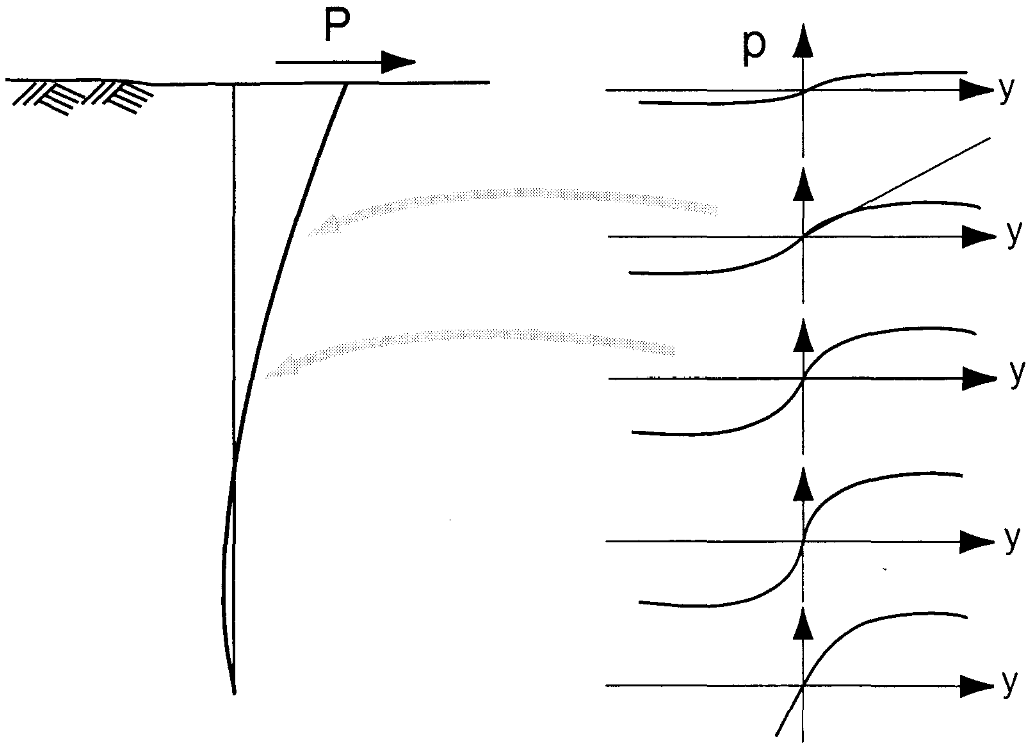
Pile Lateral Deflection

- Δ and θ Are "Coupled"

P $\rightarrow$ Δ and θ



'p-y' Relations (Curves) for Piles

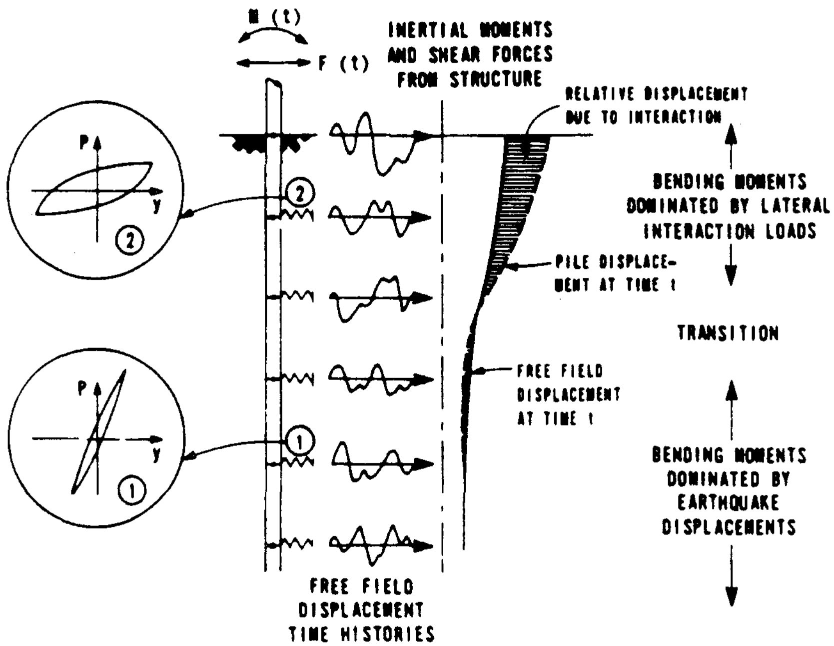


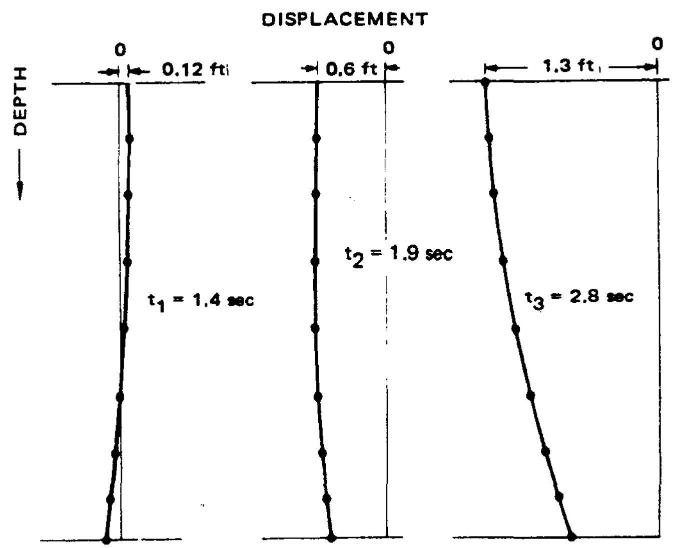
Stiffness and Lateral Capacity Increase with Depth

Lateral Deflection

Resistance of Soil at Depth

Consideration of the Free-Field Ground Motion

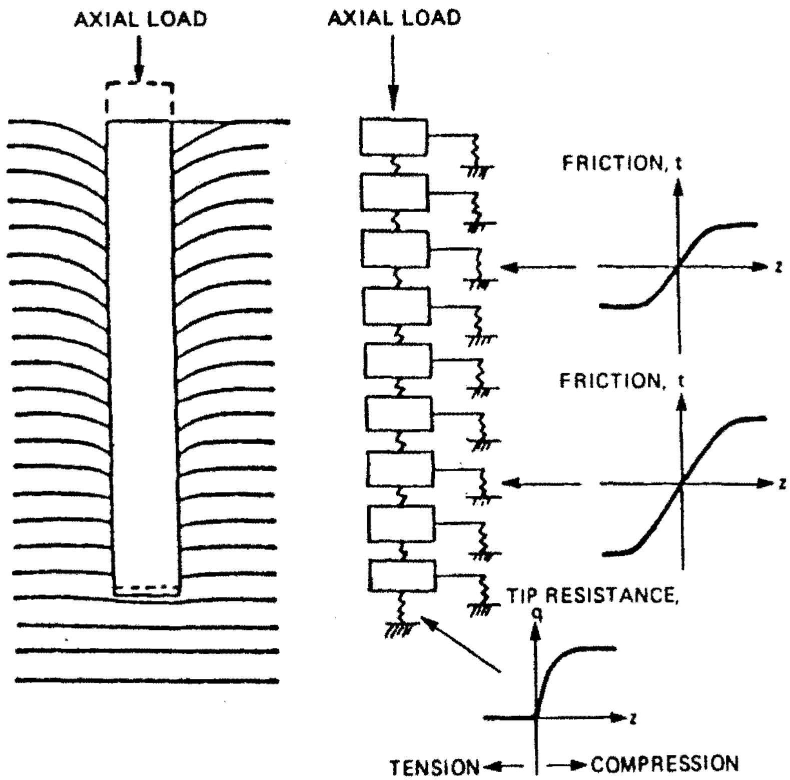




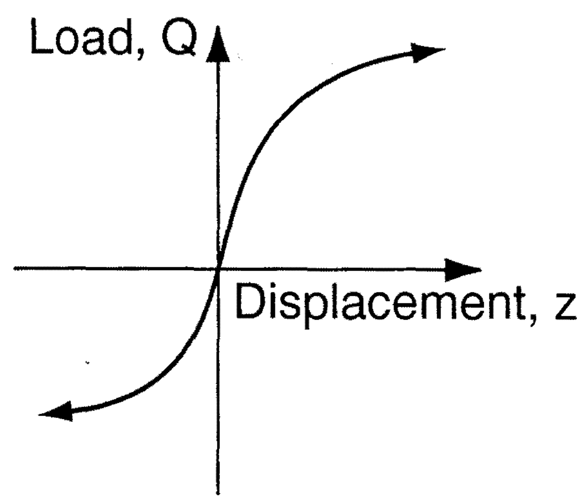
Actual Displacement Profile

AASHTO (1995)

Behavior Under Vertical Loading



FHWA (1987)

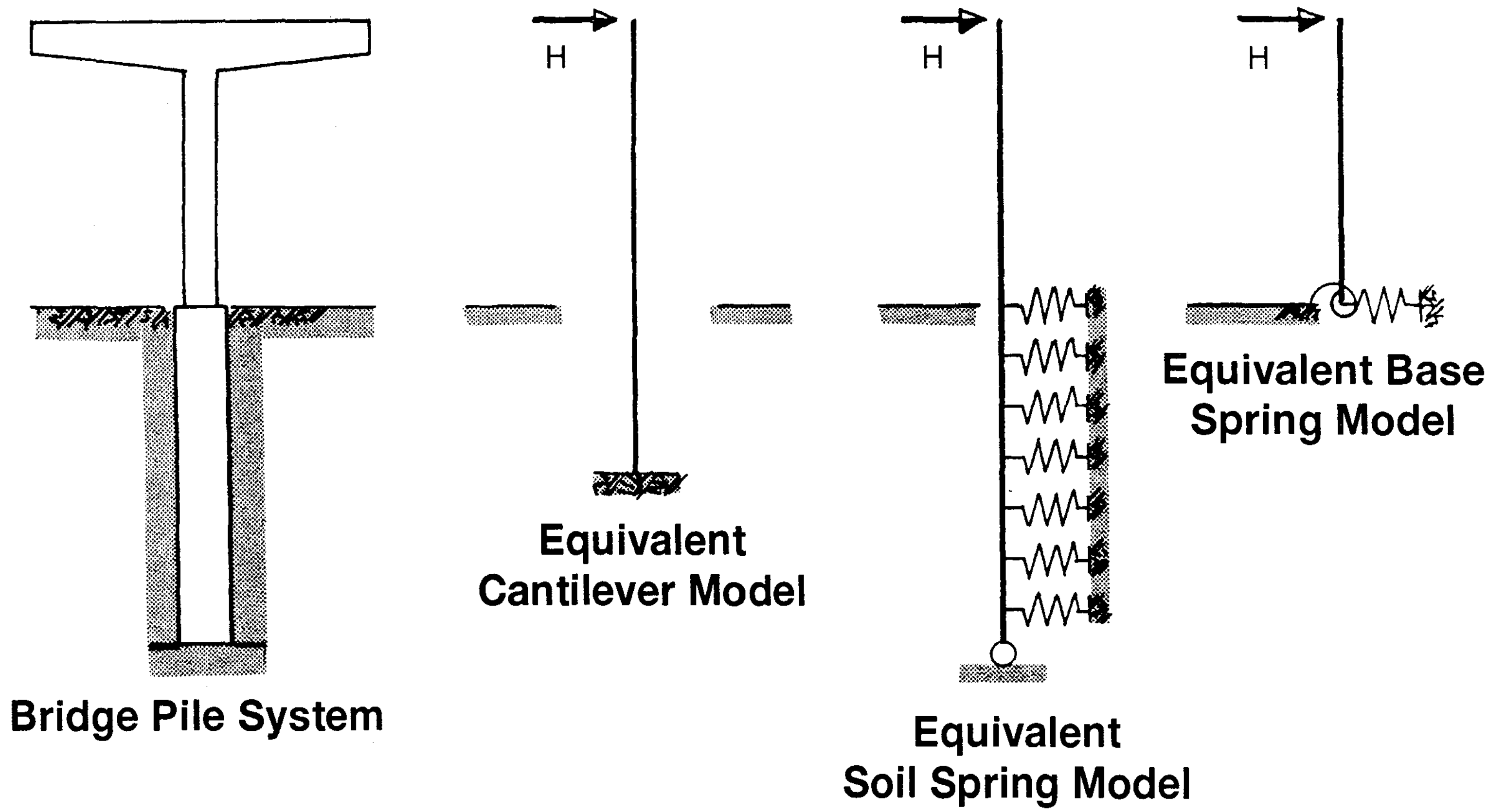


Pile-Soil System

Session 5
Piles

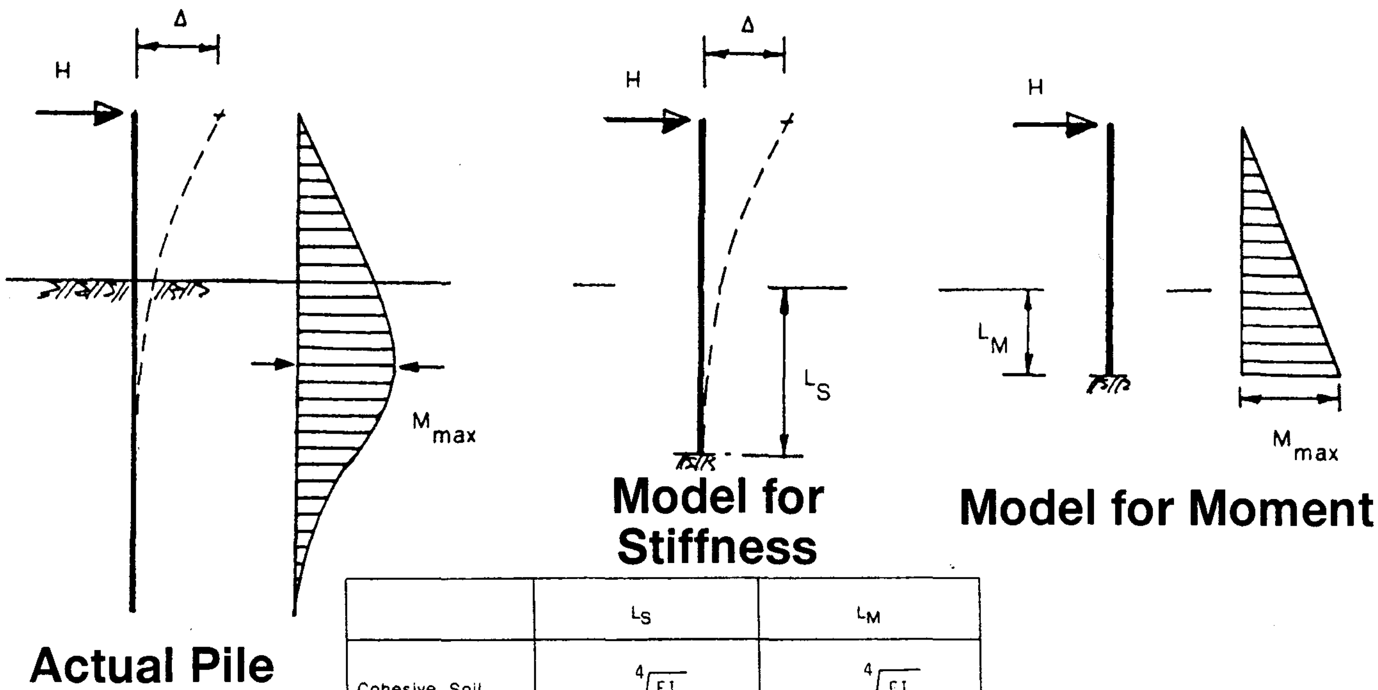
- Configuration and Behavior
- **Including Flexibility in Analysis**
- Coupling Effects
- Nonlinear Effects
- Multiple Pile Goups / Axial Stiffness
- Design and Detailing

Analytical Models of Pile Foundations



FHWA (1987)

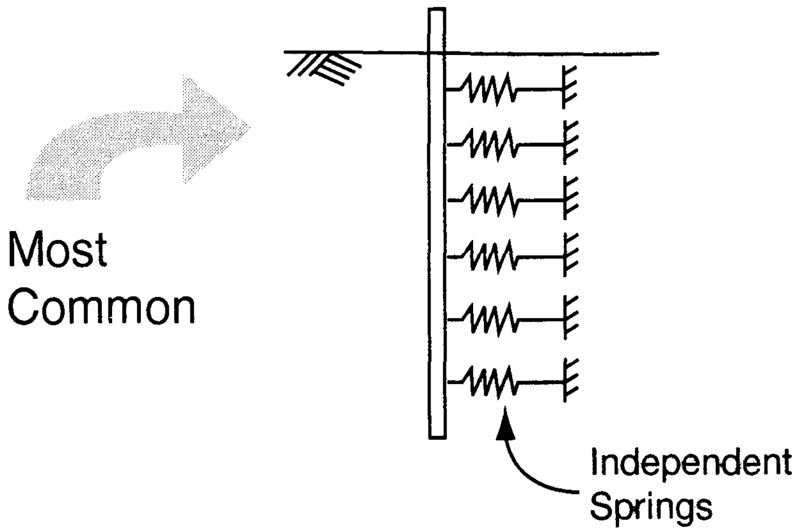
Equivalent Cantilever Method



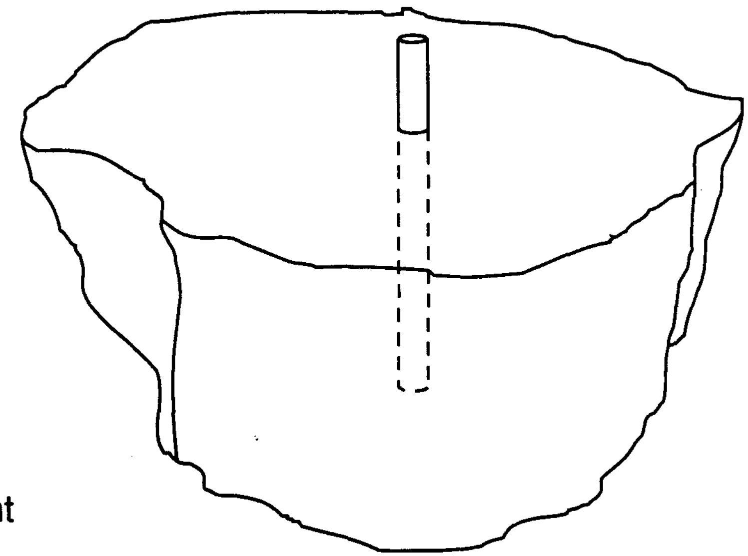
	L_S	L_M
Cohesive Soil Constant K_h	$1.4\sqrt[4]{\frac{EI}{K_h}}$	$0.44\sqrt[4]{\frac{EI}{K_h}}$
Cohesionless Soil Constant n_h	$1.8\sqrt[5]{\frac{EI}{n_h}}$	$0.78\sqrt[5]{\frac{EI}{n_h}}$

FHWA (1987)

Determining Piles-Soil Stiffness



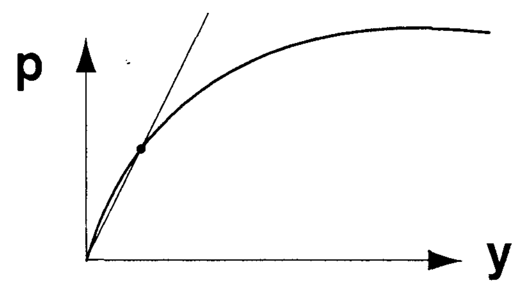
Subgrade Reaction Method



Elastic Continuum (Half-Space)

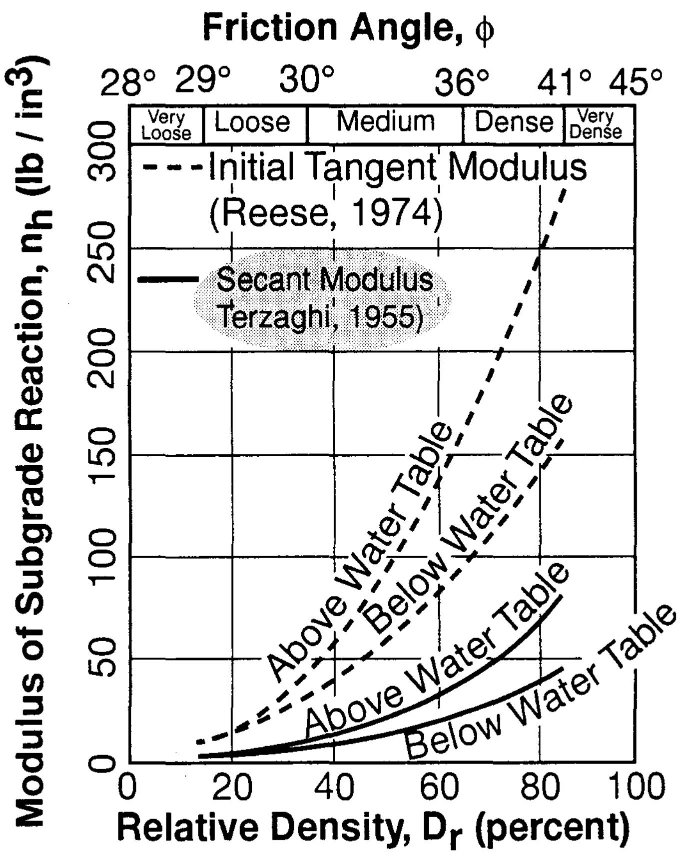
Subgrade-Reaction Method (Linear Elastic)

- **Basis** (Assumptions)
  - Known Modulus of Subgrade Reaction, n_h
  - Modulus, Function of Depth and Lateral Stiffness Is Independent of the Pile Diameter (Cohesionless and Cohesive)
  - Stiffness Typically Is Secant and Applies for About 1/3 of Ultimate Capacity



References: FHWA/RD-86/102 (1986)
NAVFAC DM7.02 (1986)
Poulos and Davis (1980)

Modulus of Subgrade Reaction / Cohesionless



- **Modulus at Depth z:**

  $k_h = n_h \frac{z}{D}$ (kip/ft^3)

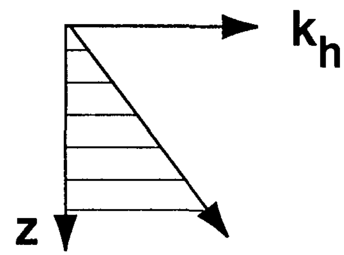
  D = Diameter

- **Spring Stiffness:**

  $K_h = k_h DH$

  H=Tributary Height

FHWA (1986)



Modulus of Subgrade Reaction / Cohesive

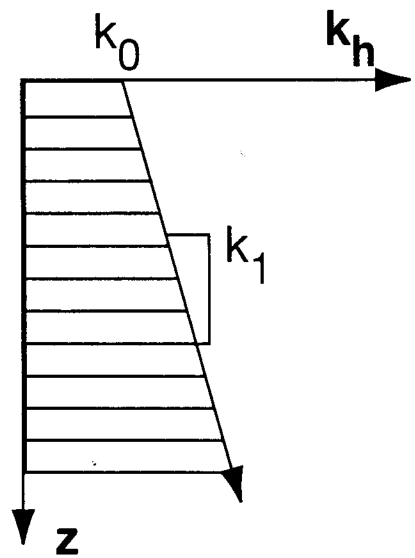
- **Modulus at Depth**

$$k_h = k_0 + k_1 z$$

$$k_0 = 0.6\, c / \varepsilon_c$$

$$k_1 = \frac{0.2}{\varepsilon_c}\left(\gamma + \frac{0.25\, c}{D}\right)$$

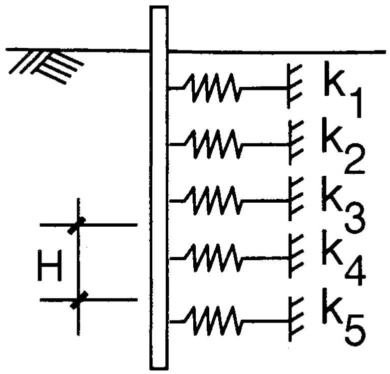
c = Undrained Shear Strength
γ = Effective Unit Weight
ε_c = Strain Amplitude at 1/2 Peak Deviatoric Stress



FHWA (1986)

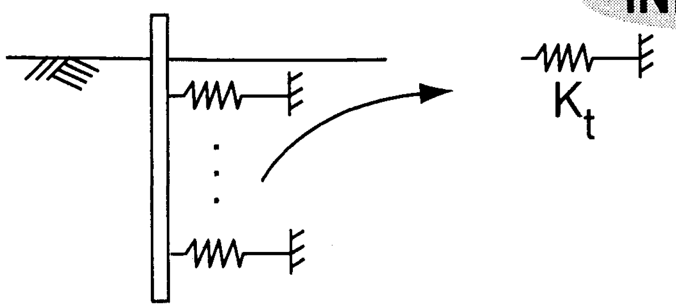
Including Stiffness

DIRECT



- Use Equations for Subgrade Method and Calculate K_1, ... etc.
- Include K's in Model Along with Pile

INDIRECT



- Use Existing (Linear Elastic) Solutions that Give Spring Stiffness at Ground Surface

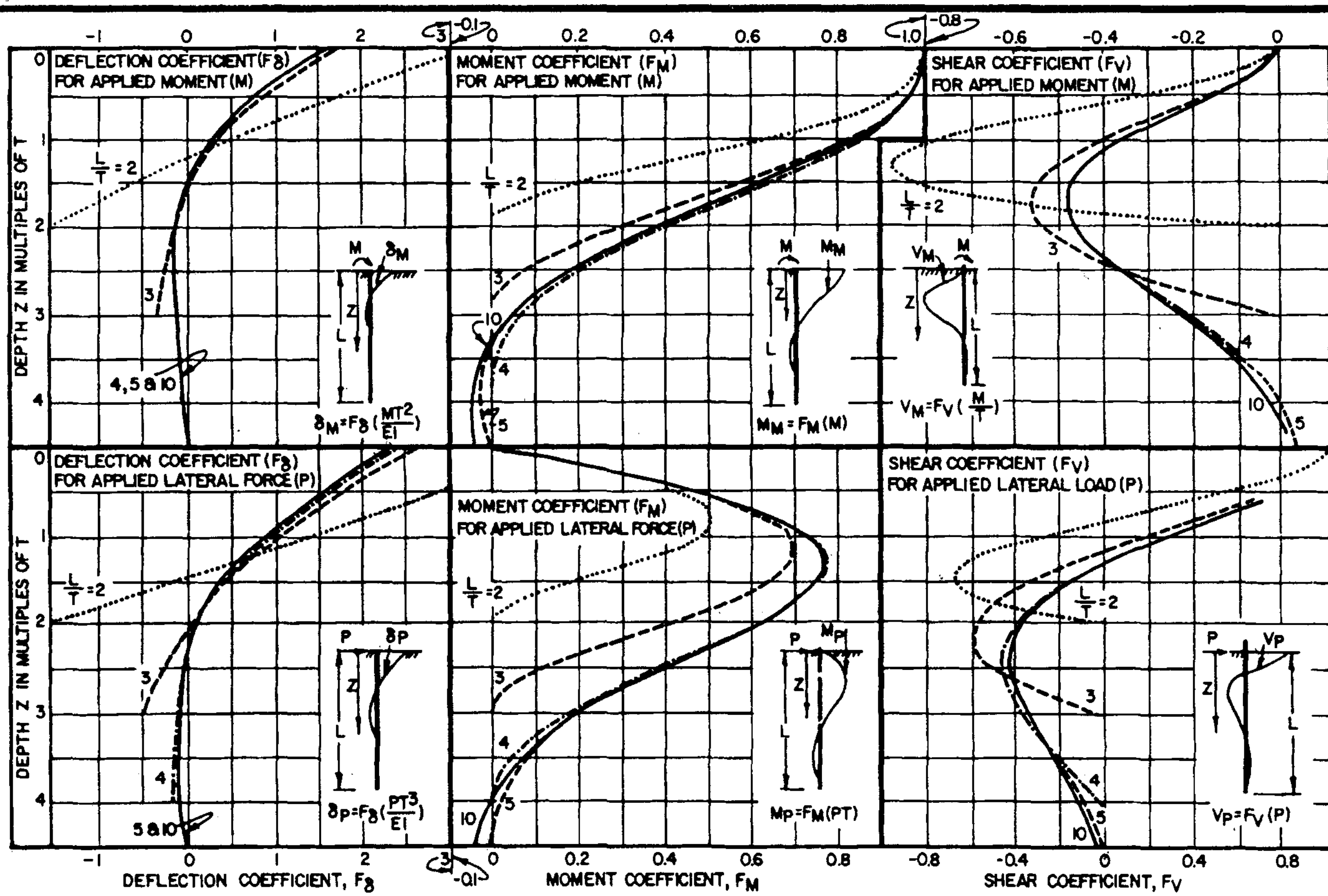
Example of 'Indirect' Method

- **Use Influence Charts (NAVFAC DM7.02, for Example)**

  1. Find n_h for Soil Type
  2. Determine Characteristic Length, $T = \left(\frac{EI}{n_h}\right)^{1/5}$
  3. Calculate L / T (L=Pile Length)
  4. Use Charts to Calculate Stiffness, Moment, and Shear — Free or Fixed Head Piles

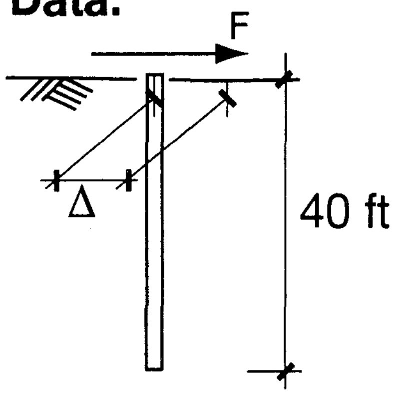
     (Use Superposition — Treat Forces and Moments Applied to Pile Separately)

NAVFAC DM7.02 Coefficients / Free Head



Example / 'DM7' Method (1 of 3)

Data:



12 in. Concrete-Filled Pipe Pile / Free Head /

$I = 406\ in^4$ (Pipe + Concrete - Upper Bound)

$E_s = 29000$ ksi

Soil (Cohesionless) $\varphi \approx 33°$ ($n_h = 23$ pci)

Required: Lateral Translational Stiffness

$$\text{Characteristic Length, } T = \left(\frac{EI}{n_h}\right)^{1/5} = \left(\frac{29000(406)}{0.023}\right)^{1/5} = 55.1 \text{ in.}$$

Example / 'DM7' Method (2 of 3)

$$\frac{L}{T} = \frac{40(12)}{55.1} = 8.7$$

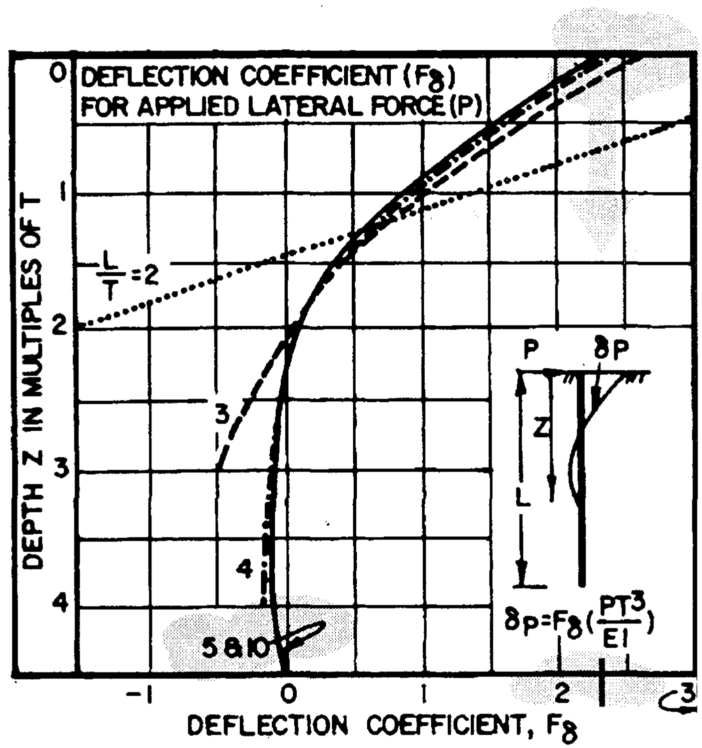
For Stiffness, Use:

$$z=0\text{ ft} \longrightarrow F_\delta = 2.3$$

$$K = \frac{P}{\delta_P} = \frac{EI}{F_\delta T^3}$$

$$K = \frac{(29000)\ 406\ (12)}{2.3\ (55.1)^3}$$

$K = 367$ kip / ft



Example / Check Using 'Equivalent Cantilever' (3 of 3)

- **Cantilever Length** $L_s = 1.8\sqrt[5]{\dfrac{EI}{n_h}} = 1.8\sqrt[5]{\dfrac{29000(406)}{0.023}}$

  $L_s = 99.3$ in.

- **Stiffness** $K = \dfrac{3EI}{L_s^3} = \dfrac{3(29000)406(12)}{(99.3)^3} = 433 \text{ kip / ft}$

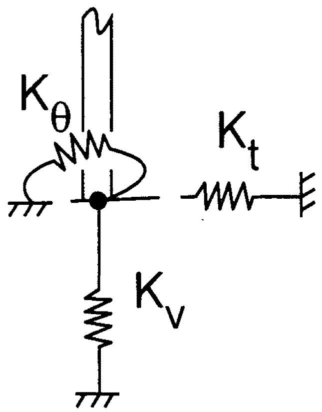
  vs. 367 kip / ft

Session 5
Piles

- Configuration and Behavior
- Including Flexibility in Analysis
- **Coupling Effects**
- Nonlinear Effects
- Multiple Pile Goups / Axial Stiffness
- Design and Detailing

Coupling Effects / Overview

No Coupling



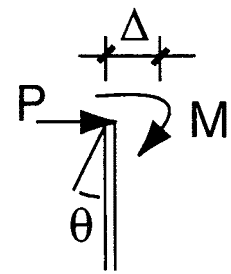
- Individual Springs

$$P = K_t\Delta_t$$
$$M = K_\theta\theta$$
$$V = K_v\Delta_v$$

Coupling (P and M)

$$P = K_{tt}\Delta + K_{t\theta}\theta$$

$$M = K_{\theta t}\Delta + K_{\theta\theta}\theta$$



- Apply P Alone $\longrightarrow$ Δ and θ
- Apply M Alone $\longrightarrow$ Δ and θ

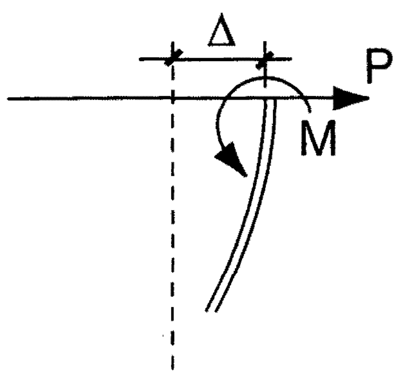
- Include in Model with Either Stiffness or Flexibility Matrix for Foundation Node

Calculating Coupled Stiffnesses (1 of 3)

- **Desired** $K_{tt}\ ,\ \underbrace{K_{t\theta}\ ,\ K_{\theta t}}_{\text{Coupling Terms}}\ ,\ K_{\theta\theta}$

- **Obtain These By**

1. Hold $\theta = 0$ / Apply $\Delta = 1$ / Calculate P and M*



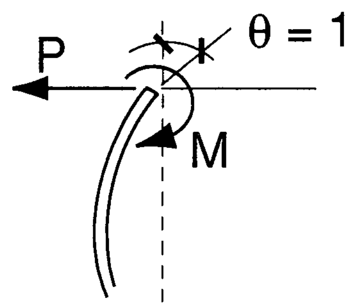
$$P = K_{tt}\overset{1}{\cancel{\Delta}} + K_{t\theta}\overset{0}{\cancel{\theta}} = K_{tt}$$

$$M = K_{\theta t}\overset{1}{\cancel{\Delta}} + K_{\theta\theta}\overset{0}{\cancel{\theta}} = K_{\theta t}$$

*Use Fixed-Head Charts Provided at End of Section

Calculating Coupled Stiffnesses (2 of 3)

2. Hold $\Delta = 0$ / Apply $\theta = 1$ / Calculate P and M



$$P = 0 + K_{t\theta}\cancelto{1}{\theta}$$

$$M = 0 + K_{\theta\theta}\cancelto{1}{\theta}$$

(See Outline of Method on Next Page)

3. Check / If Linear Elastic $\longrightarrow K_{t\theta} = K_{\theta t}$

- **Analysis Programs Use These Coefficients**
  (These Are Terms of “6 x 6 Matrix”)

Calculating Coupled Stiffnesses (3 of 3)

Outline for Calculating $K_{t\theta}$ and $K_{\theta\theta}$

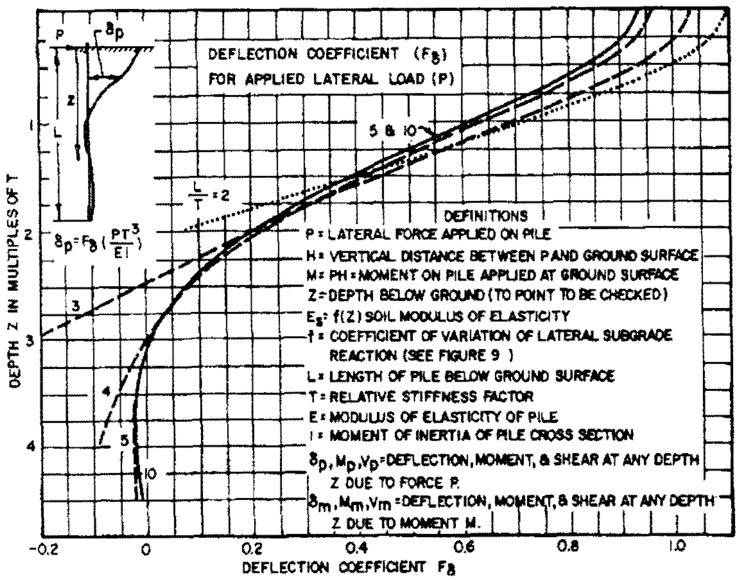
1. Apply Only P (Free Head)
   - Calculate Δ and θ (Slope) at Surface
     (Charts for Slope Given at End of Section)

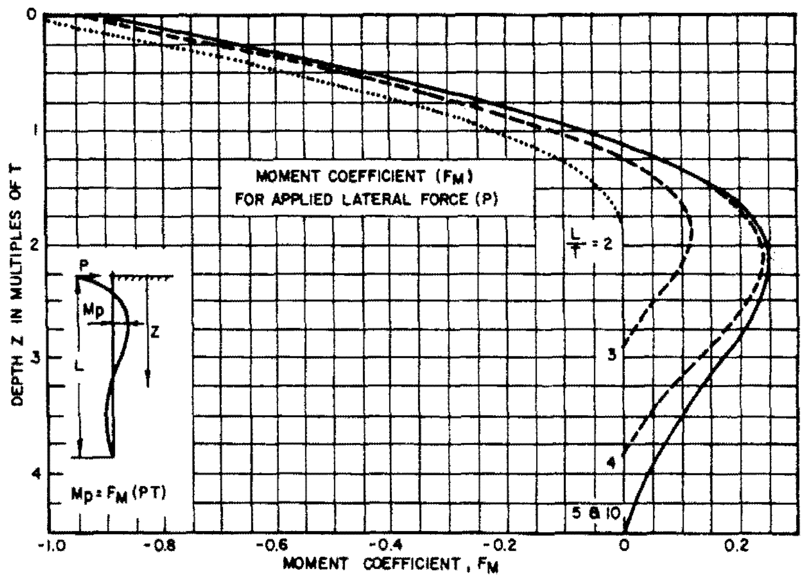
2. Apply Only M (Free Head)
   - Calculate Δ and θ at Surface

3. Form Superposition of Scaled P & M to Give $\theta = 1$ and $\Delta = 0$

Influence Coefficient / Fixed Head

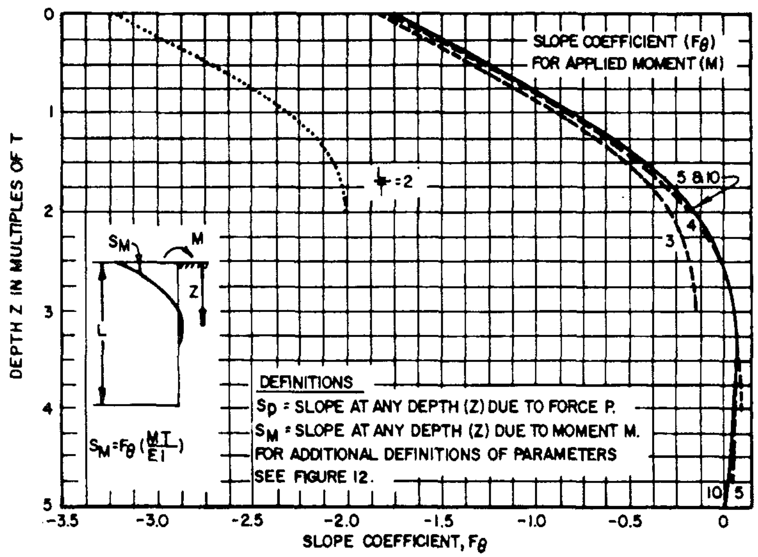
NAVFAC DM7.02 (1986)



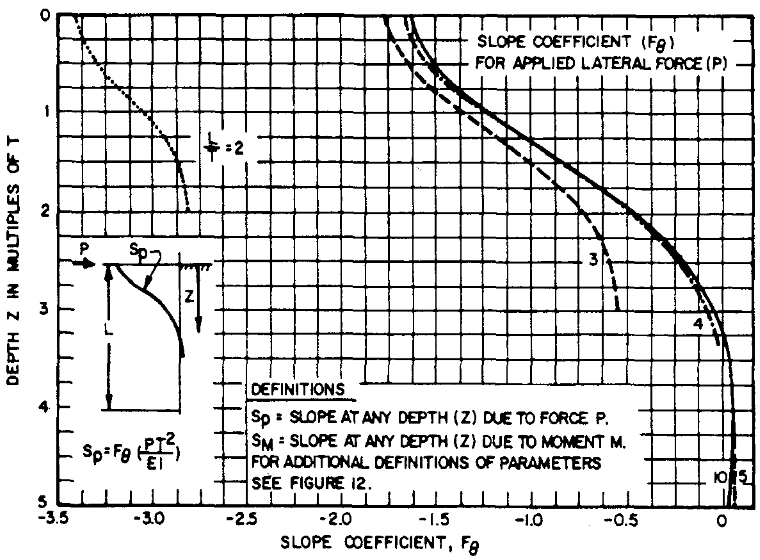


Deflection for Applied Load **Moment for Applied Load**

Slope (Rotation) of Piles / NAVFAC DM7.02



Slope for Applied Moment

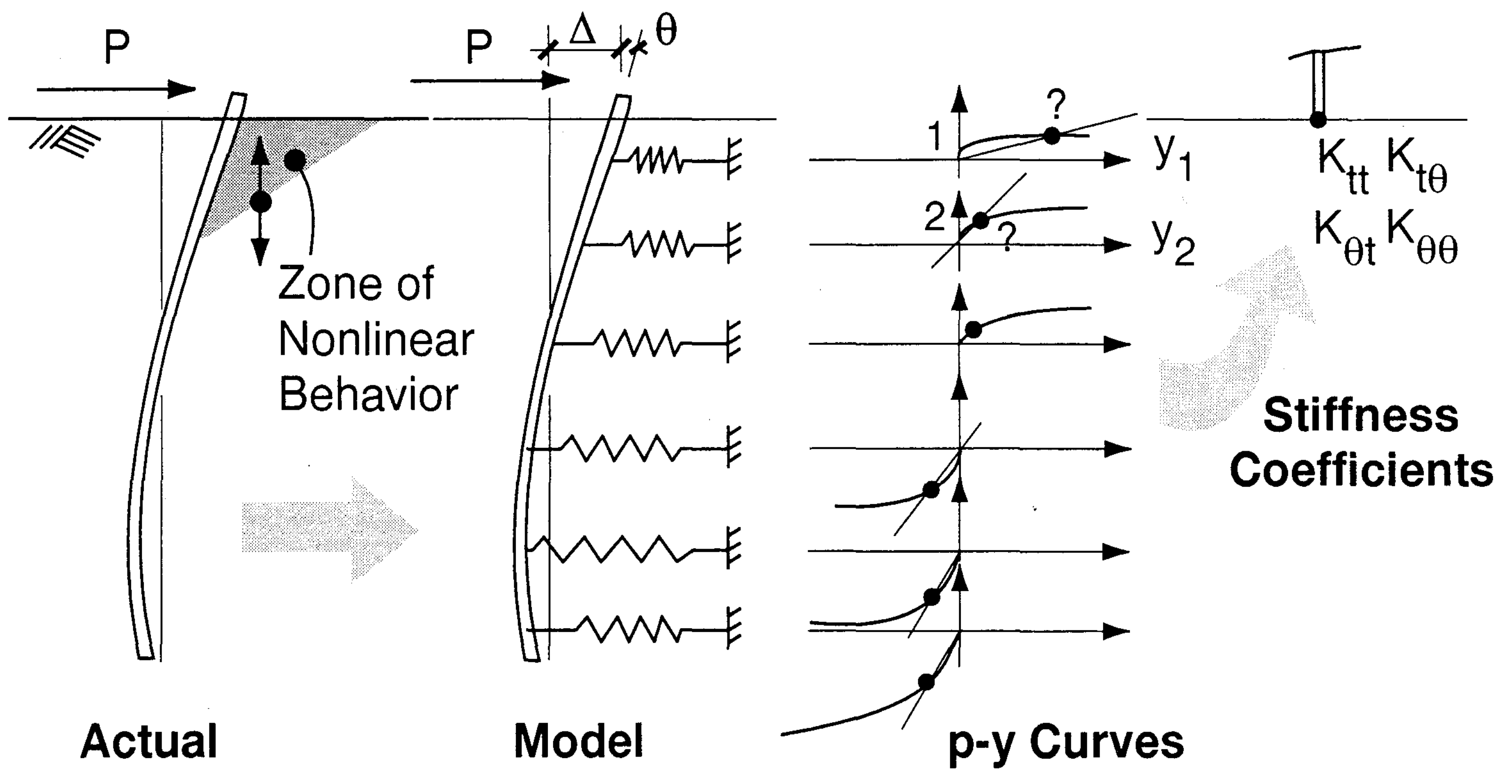


Slope for Applied Load

Session 5
Piles

- Configuration and Behavior
- Including Flexibility in Analysis
- Coupling Effects
- **Nonlinear Effects**
- Multiple Pile Goups / Axial Stiffness
- Design and Detailing

Nonlinear Effects of Soil



Developing Stiffness for Nonlinear Case

Can Use COM624 or LPile

1. Model Entire Pile
2. Generate p-y Curves for Springs Along Pile
   [FHWA (1986) or Other]
3. Load Model, Determine Δ and θ
4. For Given Δ and θ, Calculate 2×2 Matrix
5. Analyze Mathematical Model of Bridge
6. Determine New Δ and θ

Iterate Until Convergence Is Obtained

Session 5
Piles

- Configuration and Behavior
- Including Flexibility in Analysis
- Coupling Effects
- Nonlinear Effects
- **Multiple Pile Goups / Axial Stiffness**
- Design and Detailing

Effects of Closely Spaced Piles

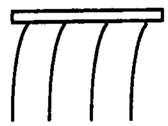
- **Group Effects**

Pile Spacing in Direction of Loading	Reduction for Subgrade Modulus, n_h
8D	1.00
6D	0.70
4D	0.40
3D	0.25

D = Pile Diameter

Stiffness for Pile Groups / Rigid Cap

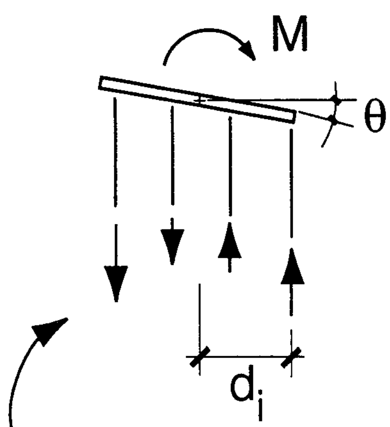
- **Translation**



$K = nK_t$

n Piles

- **Rotation**

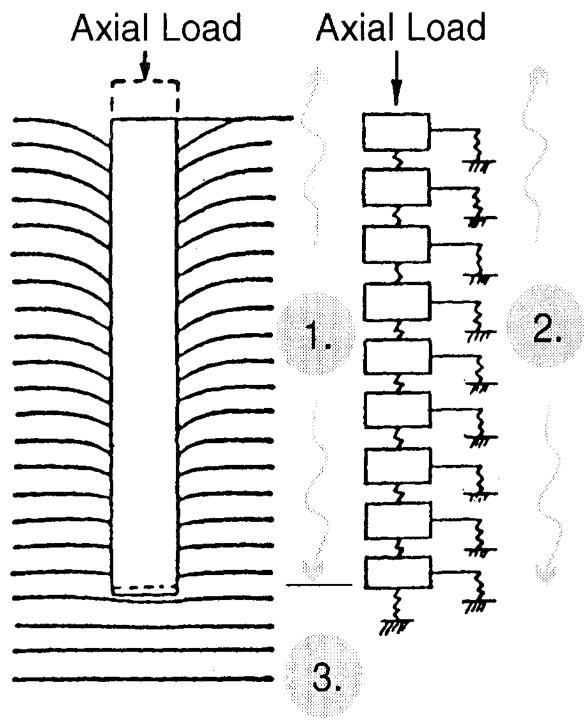


$$K_{\theta_{group}} \approx \sum_{i=1}^{n} K_{axial}\, d_i^2 = \frac{M}{\theta}$$

Axial K Most Important

Axial Stiffness Components

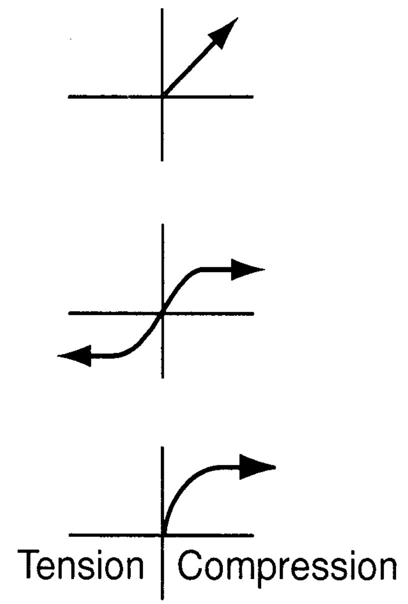
Vertical Loading Behavior



FHWA (1986)

Components of Flexibility | **Force / Deformation**

1. Pile Stiffness
2. Side Friction
3. End Bearing



Axial Stiffness of Piles

1. **Pile Stiffness** — $\frac{\text{'AE'}}{L}$

2. **Side Friction** — $f = f_{max}\left(2\sqrt{\frac{z}{z_c}} - \frac{z}{z_c}\right)$

   (No Universal Agreement, 'a Way to Do It')

f_{max} = Maximum Unit Friction

z = Slip

z_c = Critical Slip (0.2 in)

3. **End Bearing** — $q = \left(\frac{z}{z_c}\right)^{1/3} q_{max}$

   (No Universal Agreement, 'a Way to Do It')

q_{max} = Maximum Tip Resistance

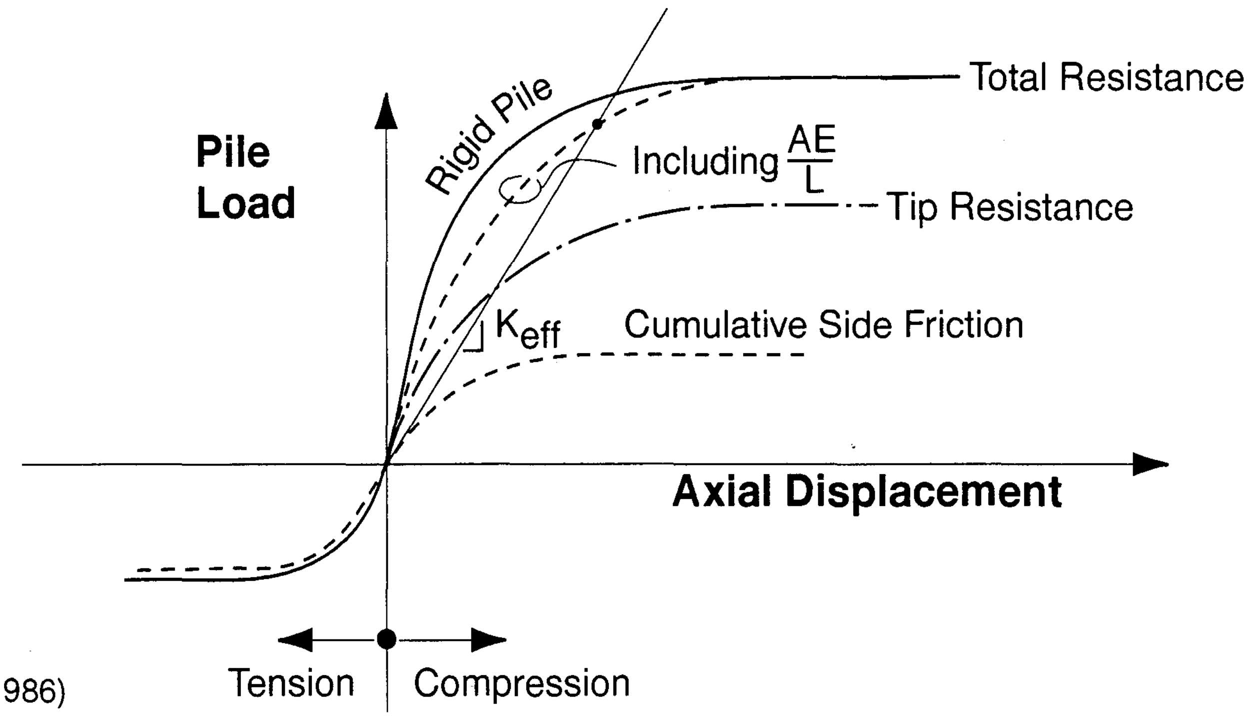
z = Deflection at Tip

z_c = Critical Displacement at q_{max} ~ 0.05 Diameter

FHWA (1986)

Axial Stiffness of Piles (continued)

Sum (Integrate) Expressions to Obtain:

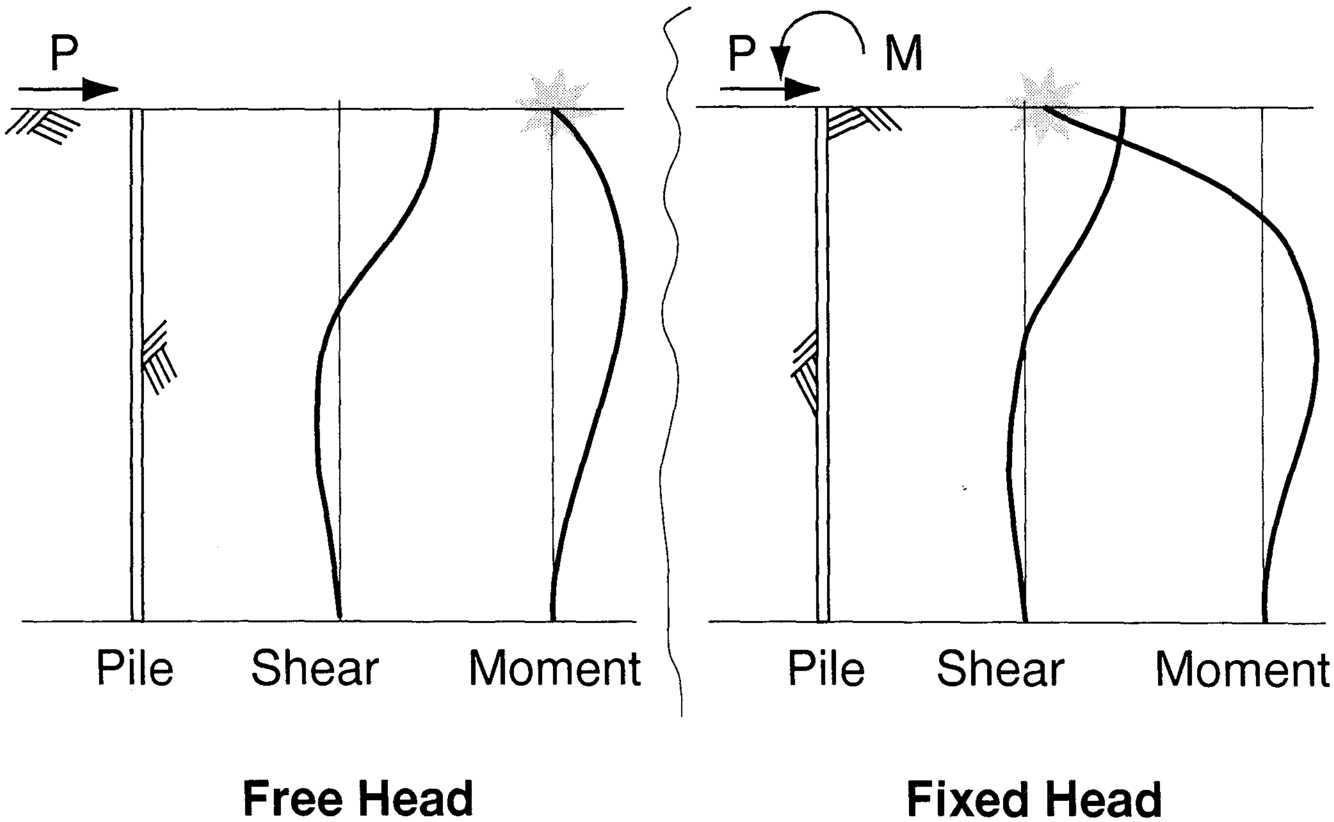


FHWA (1986)

Session 5
Piles

- Configuration and Behavior
- Including Flexibility in Analysis
- Coupling Effects
- Nonlinear Effects
- Multiple Pile Goups / Axial Stiffness
- **Design and Detailing**

Internal Force Distributions (Elastic)



Effect of Head Condition

- **Performance Objective** → Damage Should Be Detectable
  ∴ Not in Foundation
- **Design** → Elastic or Plastic Hinging Forces

Fixed Head — Large Moment / Concentrated Near Top of Pile
∴ Potential for Plastic Hinging

Free Head — Largest Moment at Depth / Distributed Curvatures

Division I-A Requirements (1 of 6)

Overview

- Capacity Protect / R = 1.0 or Hinging Forces
- Tie Piles and Cap Together
- Provide Ductility at Top of Pile

SPC B / 6.4.2 (C)

- Design to Carry All Forces

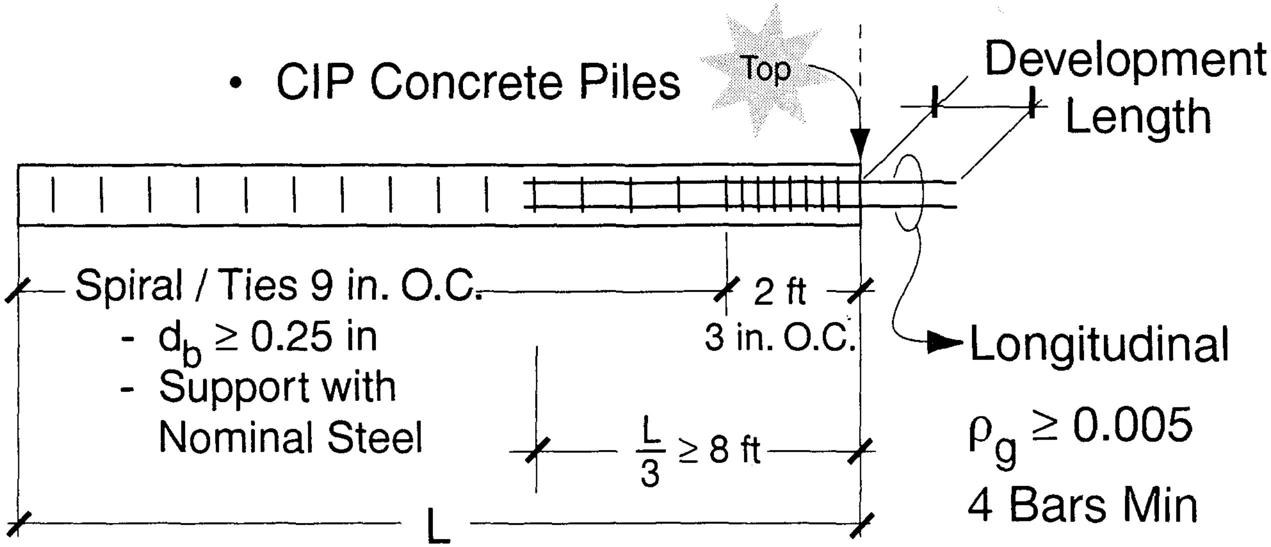
Plus

- Timber and Steel — Uplift Capacity ≥ 10% of Allowable Pile Load

Division I-A Requirements (2 of 6)

SPC B / 6.4.2 (C) (continued)

- Concrete-Filled Pipe Pile — 4 Dowels / $\rho = 0.01$
  (Note: Completely Free Head Not Realistic)
- CIP Concrete Piles



Division I-A Requirements (3 of 6)

SPC B / 6.4.2 (C) (continued)

- Precast Piles
  - $\rho_g \geq 0.01$ (4 Bars Min) Over Entire Length
  - Spiral / Ties $\geq$ #3
  - Spacing as for CIP Piles
- Precast — Prestressed Piles
  - Same Ties as for Precast Piles

Division I-A Requirements (4 of 6)

SPC C and D / 7.4.2 (C)

- Same as SPC B

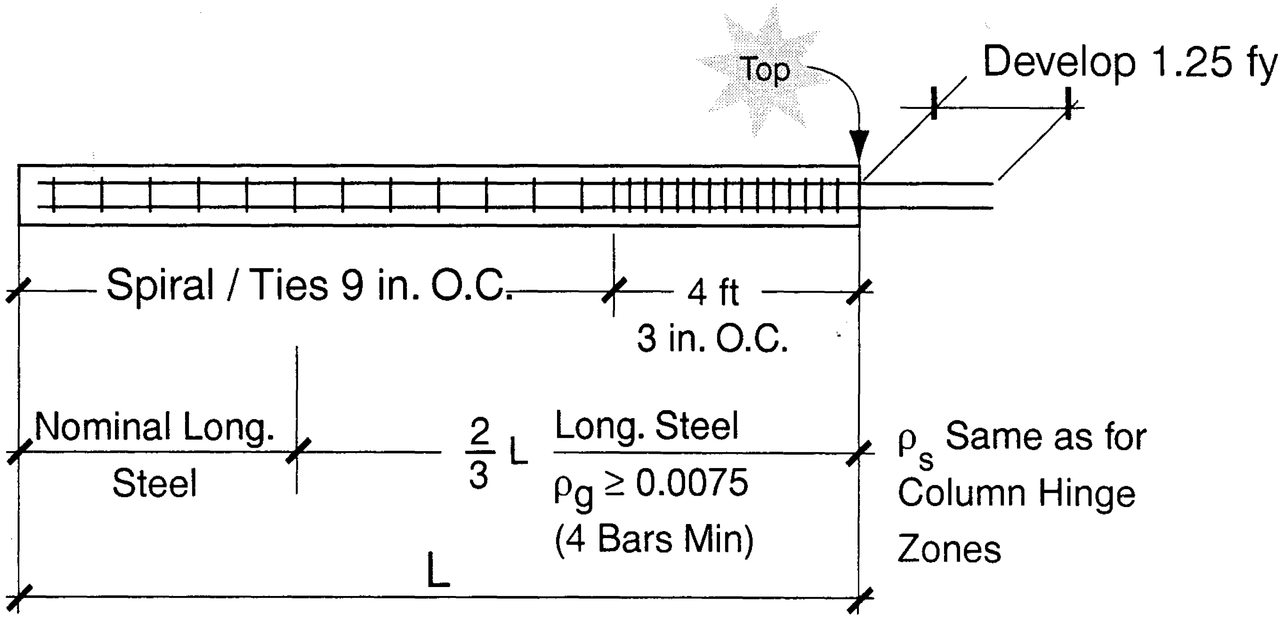
- Concrete Piles
  - Anchor to Cap to Develop 1.25 f_y of Pile Longitudinal Bars

- Potential Plastic Hinge Zones
  - Same Confinement as for Columns!
  - $2D_{pile}$ or 24 in. at Top or Other Possible Hinge Zones

Division I-A Requirements (5 of 6)

SPC C and D / 7.4.2 (C) (continued)

- CIP Concrete Piles

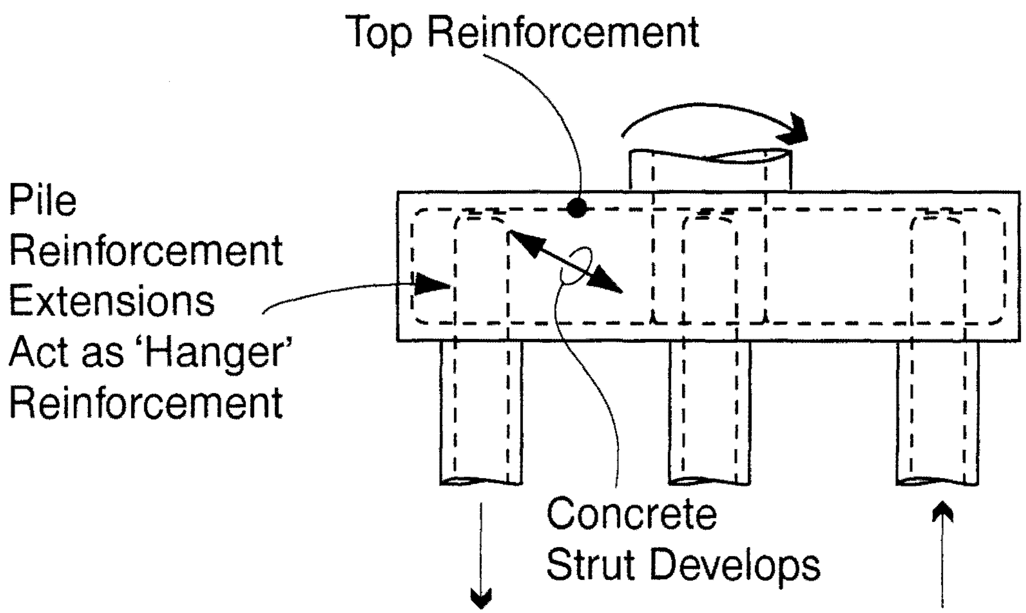


Division I-A Requirements (6 of 6)

SPC C and D / 7.4.2 (C)

- Precast Piles
  - $\rho_g \geq 0.01$ (4 Bars Min) Over Entire Length
  - Spiral / Ties $\geq$ #3
  - Spacing as for CIP Piles
- Precast — Prestressed Piles
  - Same Ties as for Precast Piles

Pile Cap Considerations for Uplift Forces



Note: Struts May
Become Quite Flat
Leading to High Forces

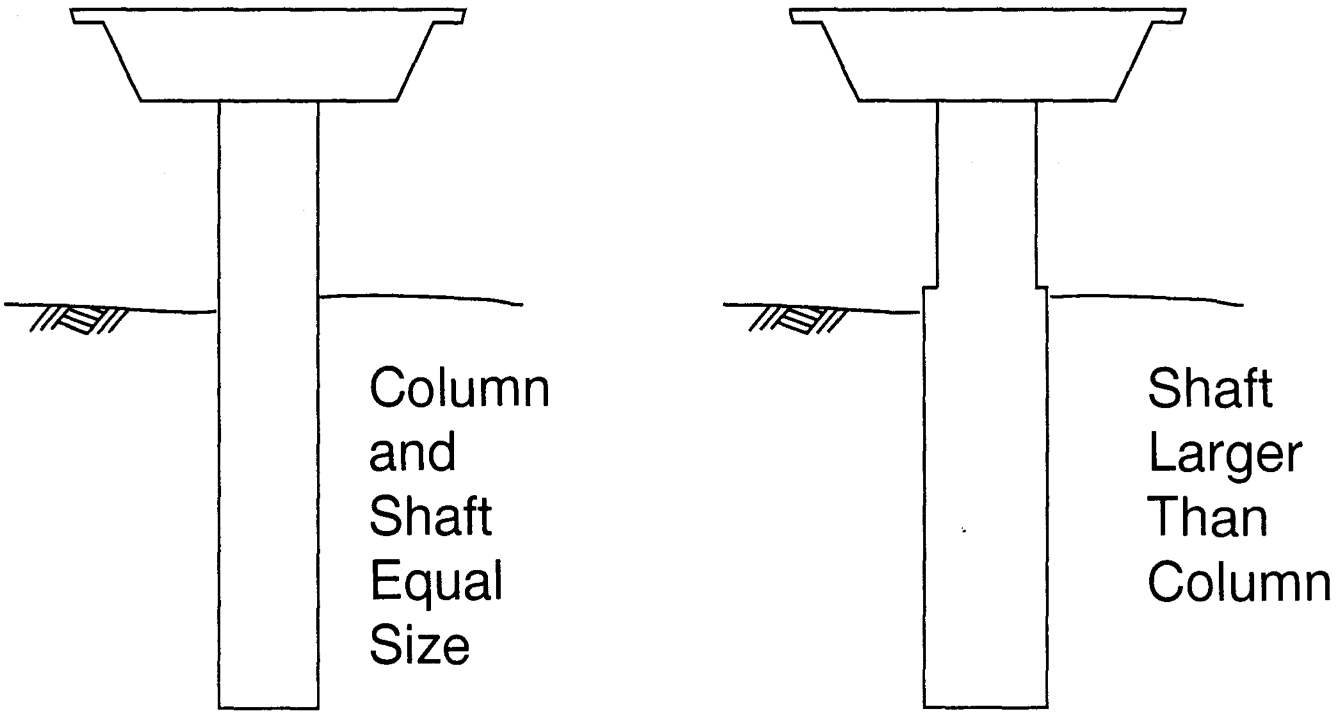
∴ May Require Stirrups

Session 6
Curved Box Girder Bridge Example
Drilled Shaft*

- **Behavior and Stiffness**
- **Design and Detailing**

* Also Called Pile Shafts, etc.

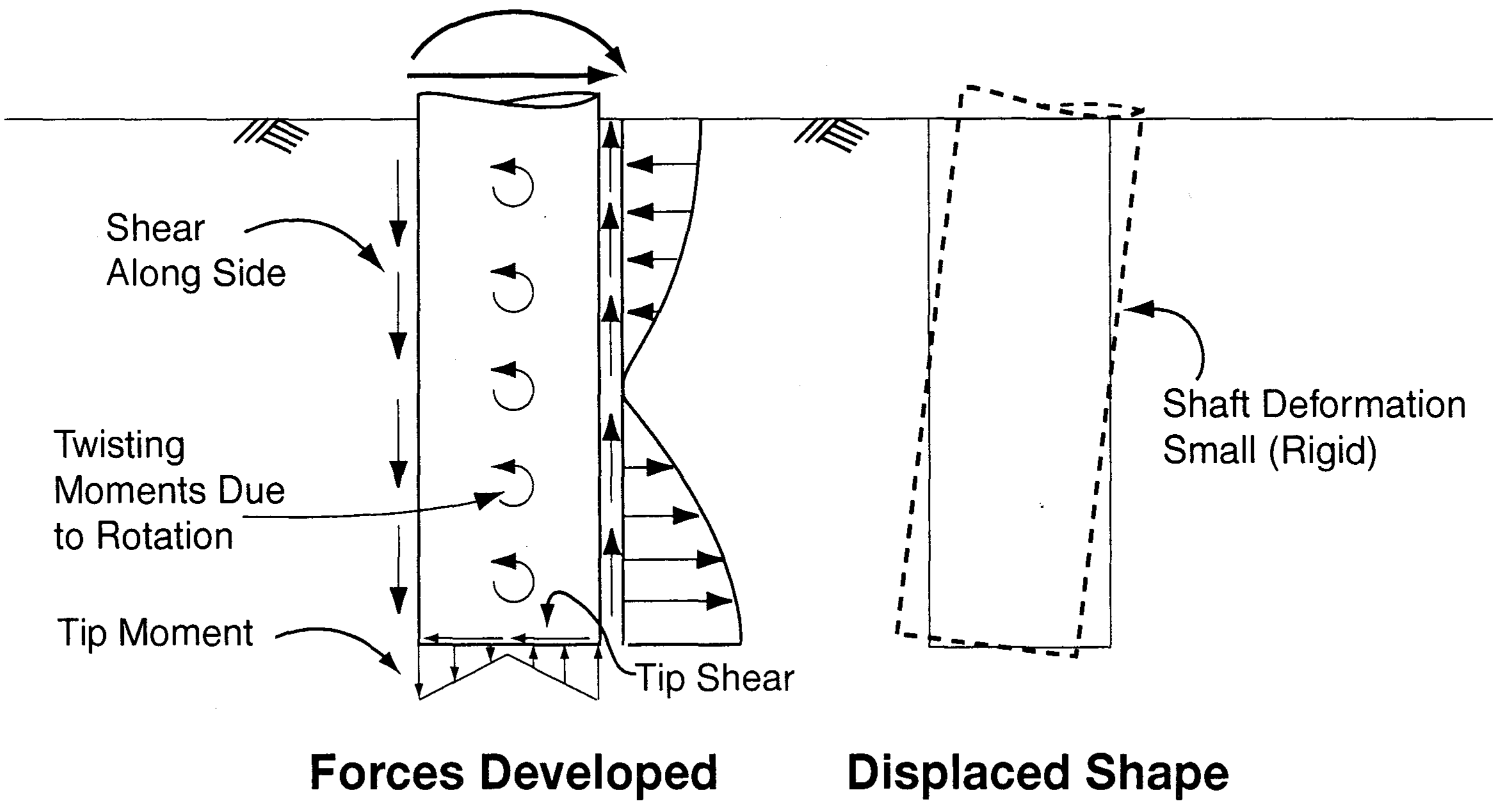
Configurations



Drilled Shaft Behavior

- Lateral Behavior Similar to Piles
- Length / Diameter (or L / T) Smaller Than Piles
  - Stiffness Less Than Longer Elements of Same Diameter
  - Lateral Stiffness More Sensitive to (L / T)
  - Coupling Between Displacement and Rotation More Important
- Larger Diameters Lead to Additional Mechanisms for Resistance

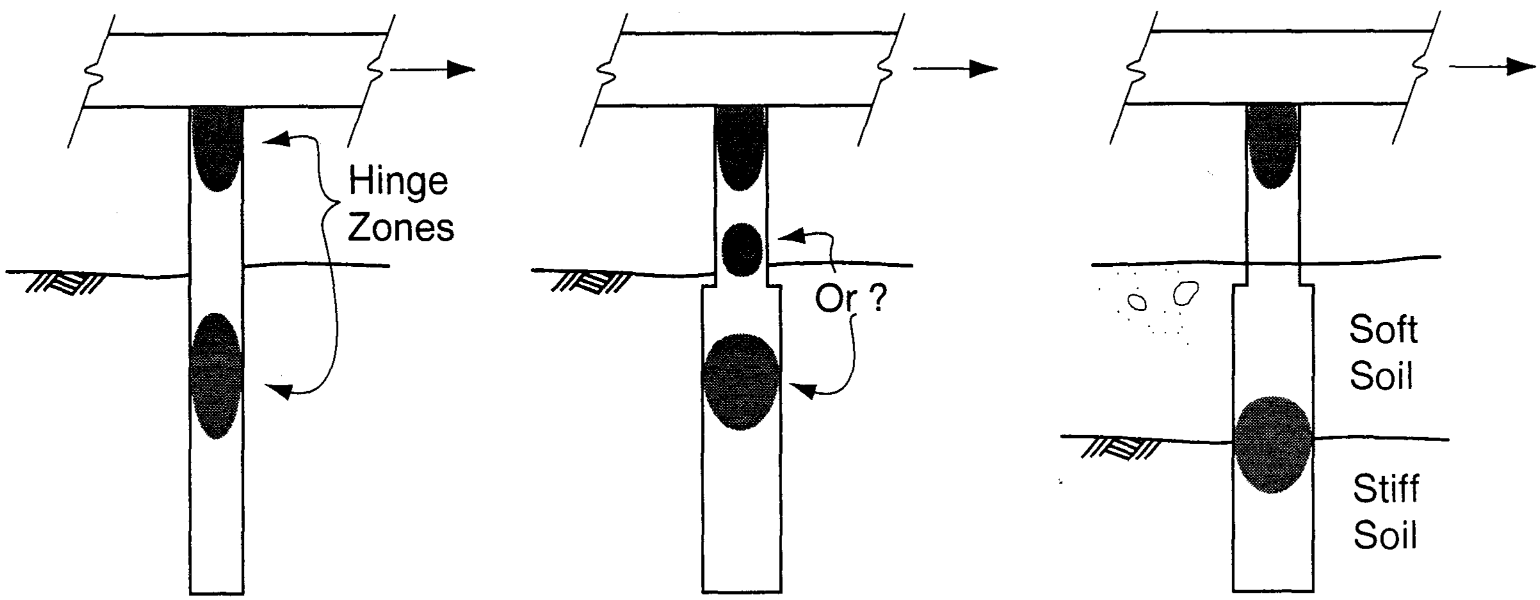
Mechanisms of Lateral Resistance



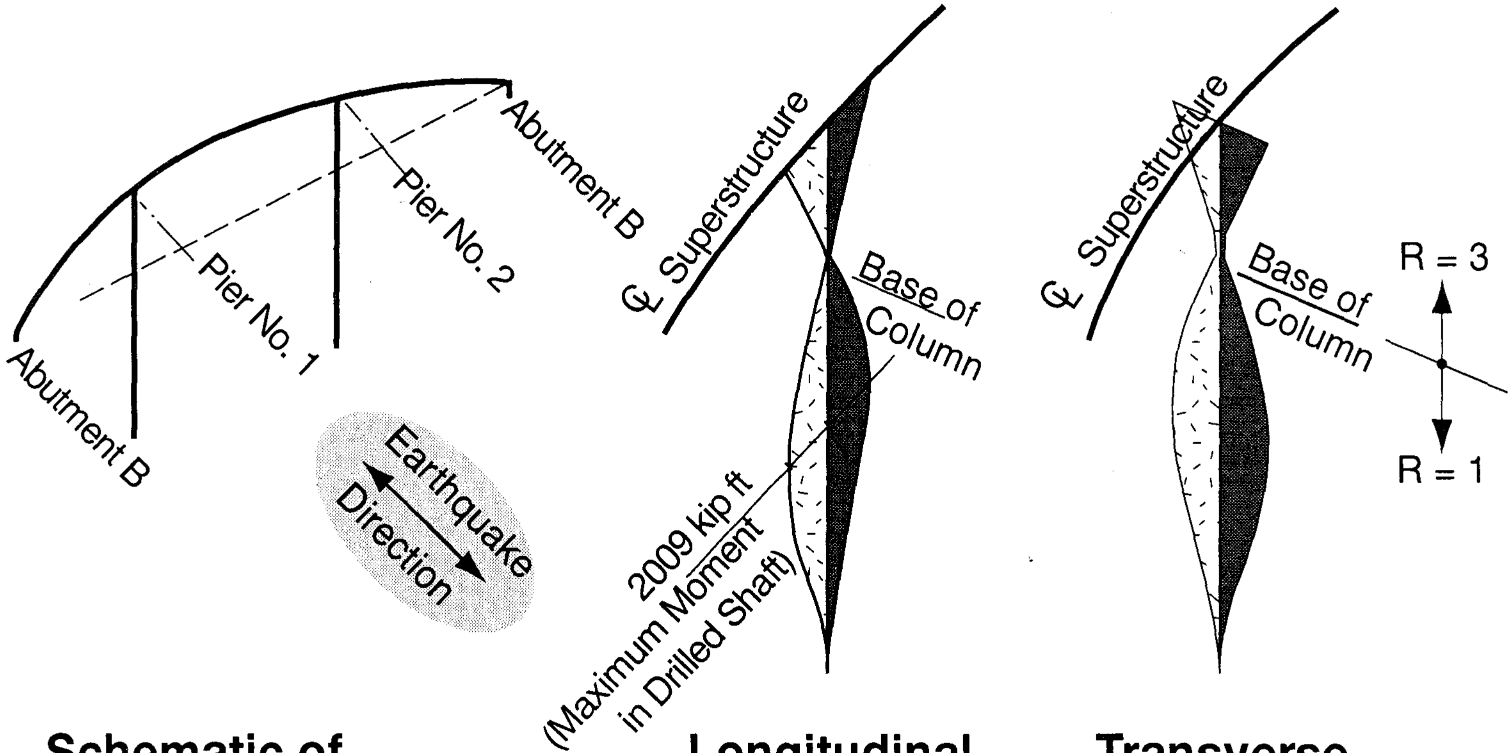
Developing Stiffness of Drilled Shafts

- Use Same Approach as for Piles
- Neglect Additional Resistance Mechanisms
  (May Underpredict Strength)
- Include Coupling Effects (More Critical Than with Piles)
- Some Methods Are Under Development for Including All Resistance Mechanisms
  (Approaches May Change in the Future)

Plastic Hinging Behavior

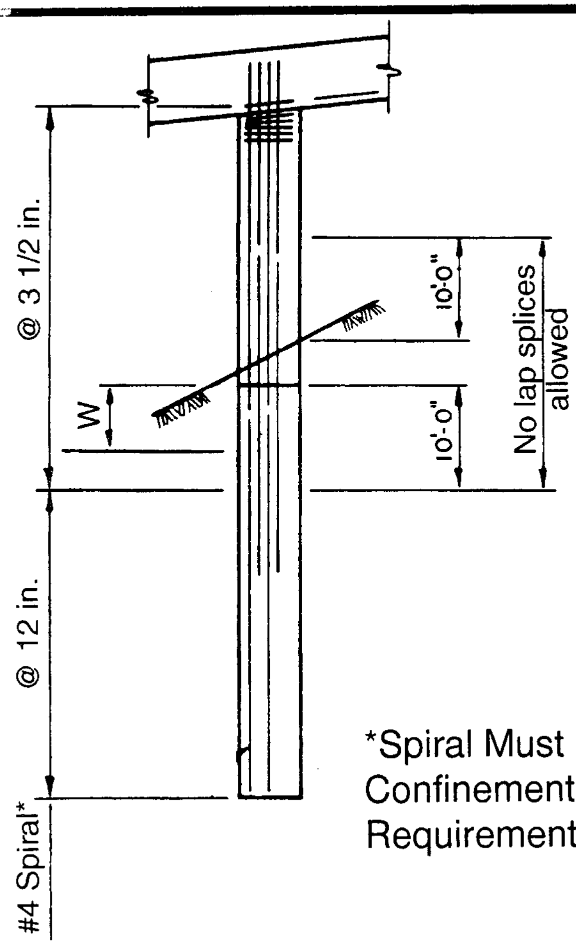


Example / Distribution of Elastic Moments

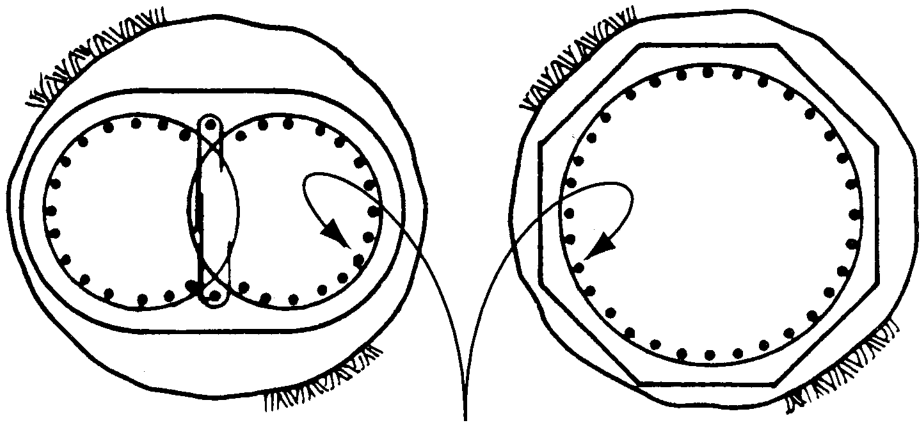


Schematic of Model **Longitudinal Moment** **Transverse Moment**

Detailing Issues/
'Same Size' Columns and Shafts



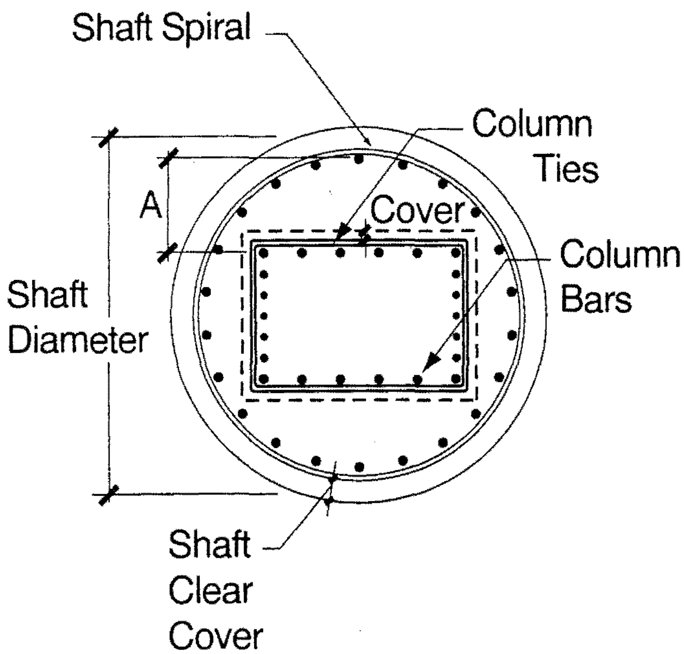
*Spiral Must Comply with Confinement and Shear Requirements



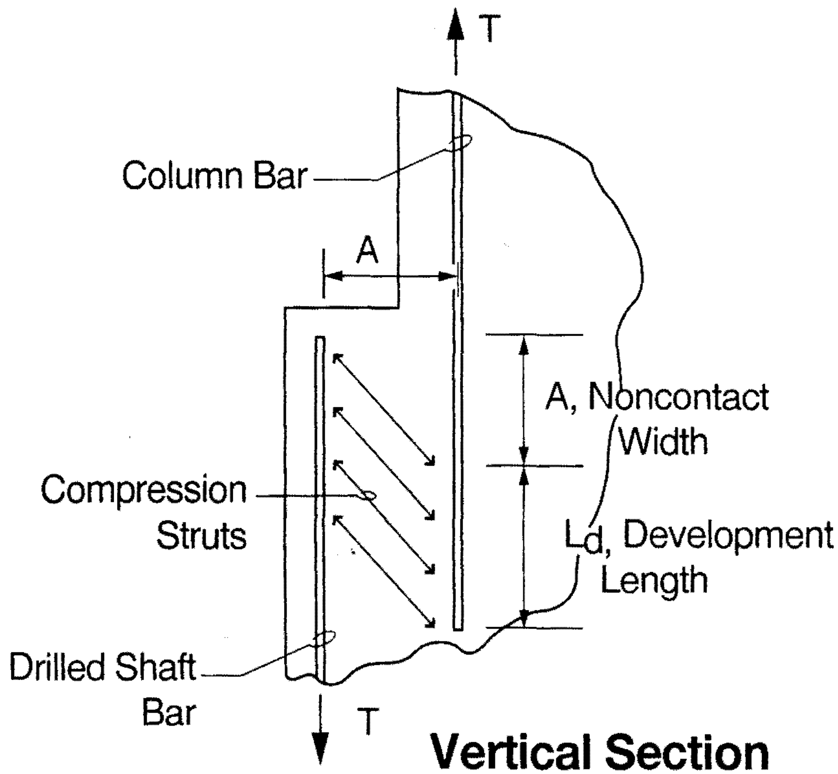
Reinforcement Pattern Extending into CIDH Pile to Be the Same as in Column

Caltrans (1995)

Detailing Issues/ Shafts Larger Than Columns

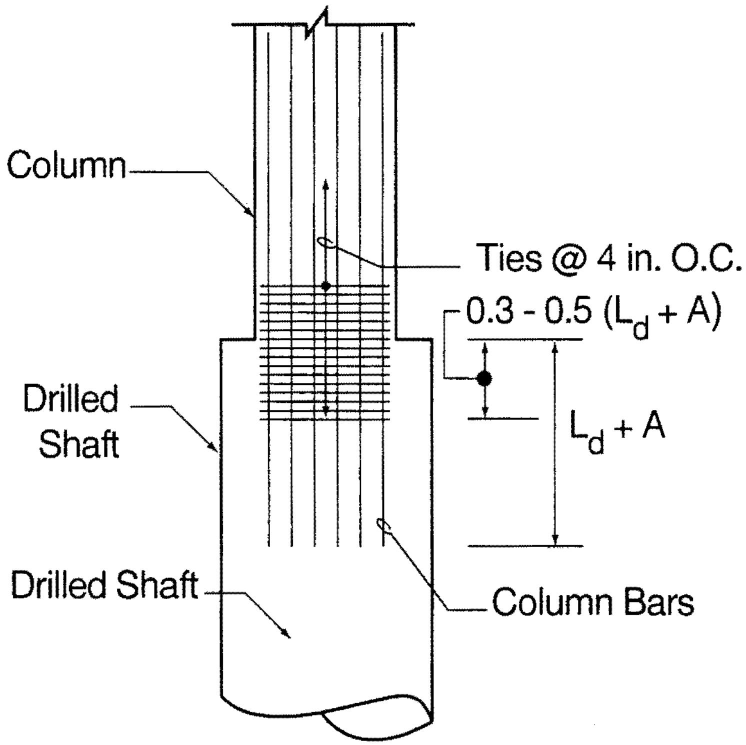


Horizontal Section

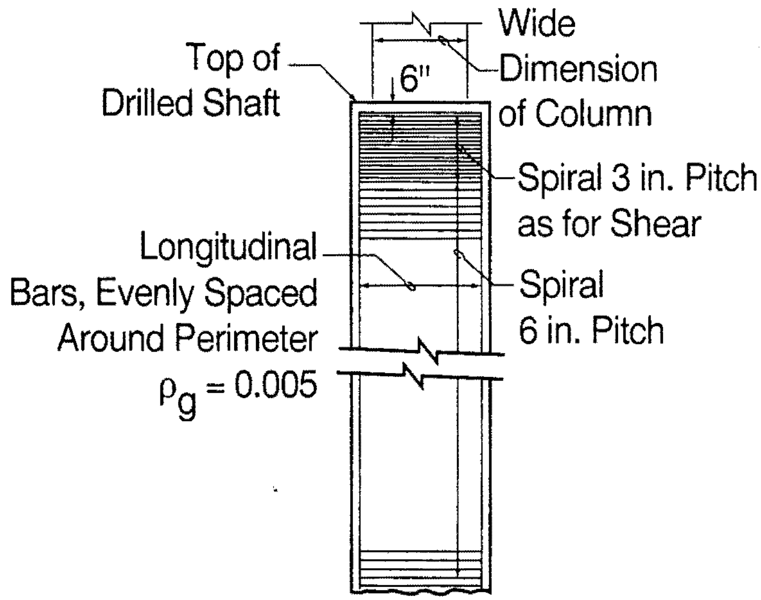


Vertical Section

Detailing Issues/ Shafts Larger Than Columns



Section at Connection

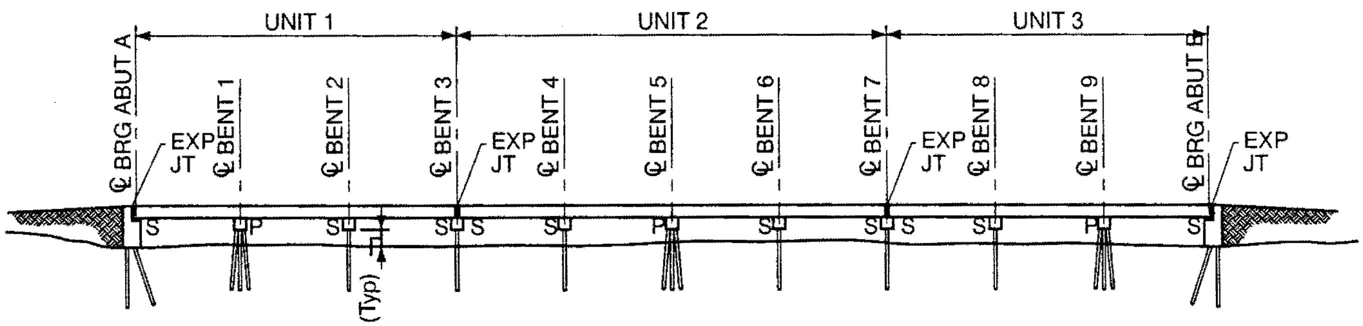


Shaft Reinforcement

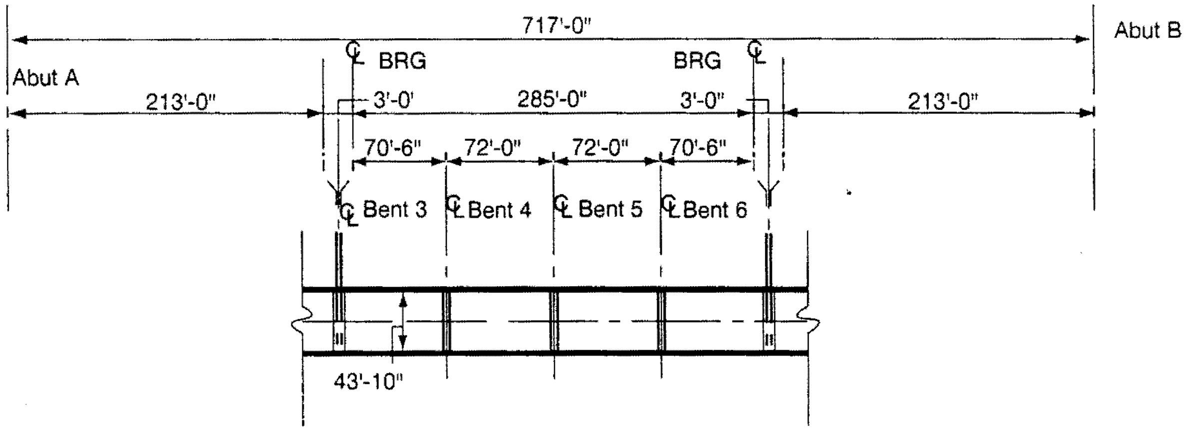
Session 6
Pile Bent Bridge Example
Pile Bent Issues

- Description
- Behavior
- Stiffness Considerations
- Design Considerations

Pile Bent Bridge / Layout and Elevation

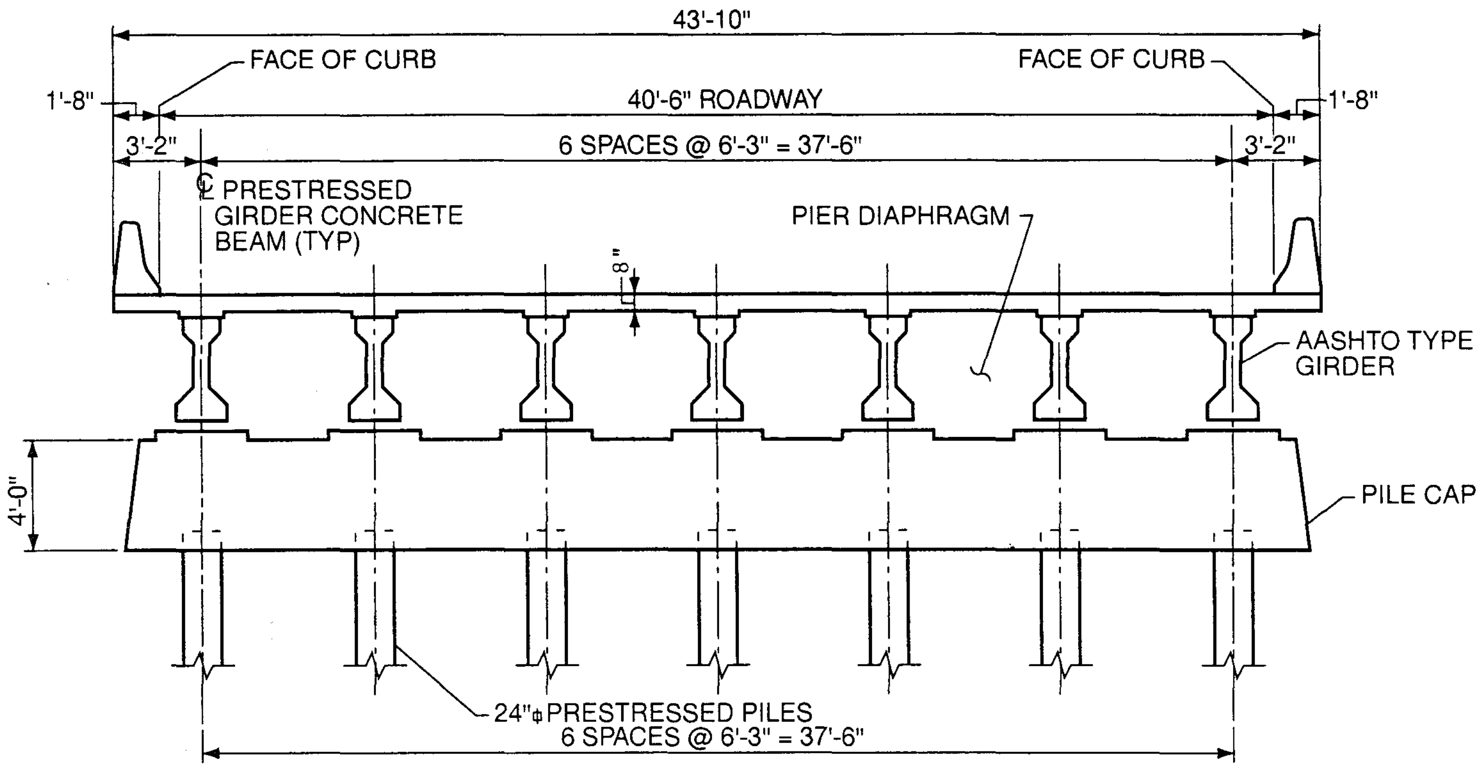


Elevation



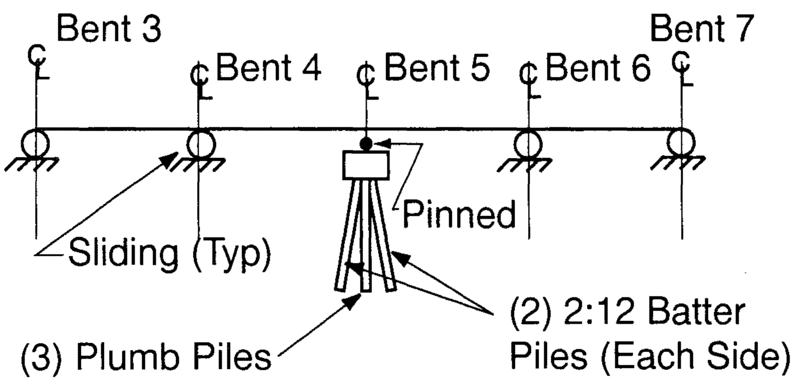
Plan of Center Unit

Pile Bent Bridge / Bent Elevation



Section

Typical Configuration / Lateral Load Transfer



Longitudinal Structural Model

- All Longitudinal Inertial Loads Taken by Bent No. 5
- All Other Bents Assumed to Have Sufficient Seat Widths
- Stiffness of and the Load Taken by Bent No. 5 Very Dependent on Number and Slope of Batter Piles

Developing the Stiffness of Pile Bents

Plumb Piles →

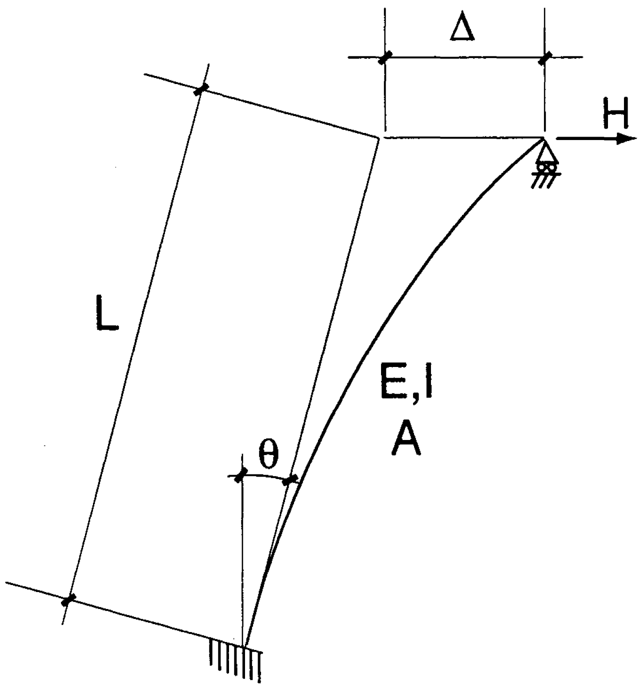
- Methods for Piles (Previously Discussed) May Be Used
- Account for Clear Height Above Mudline

Battered Piles →

- Separate Flexural and Axial Effects
- Standard Pile Methods for Flexure
- Axial Stiffness and Capacity Much More Important

Lateral Stiffness of Battered Pile

Consider One Pile of a Two Battered Pile Pair

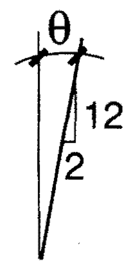


$$K = \frac{H}{\Delta} = \frac{3EI}{L^3}\cos^2\theta + \frac{AE}{L}\sin^2\theta$$

- No Rotational Restraint at Cap
- If Cap Fixed $3 \longrightarrow 12\frac{EI}{L^3}$
- No Axial (Soil) Deformation Below Pile
- If Add Flexibility Beneath Pile

$$\frac{AE}{L} = K_{eff} = \frac{1}{{}^{1}/(AE/L) + {}^{1}/K_{soil}}$$

Example / Lateral Stiffness of 2:12 Batter Piles (1 of 3)



- 24 in. Square Prestressed Concrete Pile

$E = 4030$ ksi $\quad L = 60$ ft $\quad \theta = 9.46°$
$A = 40\ \text{ft}^2 \quad I = 1.33\ \text{ft}^4$

- Use Different Effective Length to Fixity for Flexure and Axial Contributions

$L_f = 25$ ft — Based on Equivalent Cantilever for Plumb Pile
$L_a = 41.7$ ft — Based on Skin Friction and No Tip Displacement

Example / Lateral Stiffness of 2:12 Batter Piles (2 of 3)

- Flexural Contribution to Lateral Stiffness

$$K_f = \frac{3EI}{L_f^3}\cos^2\theta + \frac{3(4030)144}{(25)^3}\cos^2(9.46°) = 144 \ \frac{\text{kip}}{\text{ft}}$$

- Axial Contribution to Lateral Stiffness

$$K_a = \frac{AE}{L_a}\sin^2\theta = \frac{4.0(4030)144}{41.7}\sin^2(9.46°) = 1504 \ \frac{\text{kip}}{\text{ft}}$$

Even @ 2:12 $\quad K_a \sim 10\, K_f$

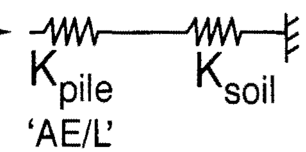
Example / Lateral Stiffness of 2:12 Batter Piles (3 of 3)

- Include (Approximately) the Surrounding Soil Flexibility

  From Geotech: Soil $\Delta \sim 0.25$ in at 600 kip maximum load

$$K_{soil} = \frac{600}{0.25} = 2400 \text{ kip/in}$$

- Assume →



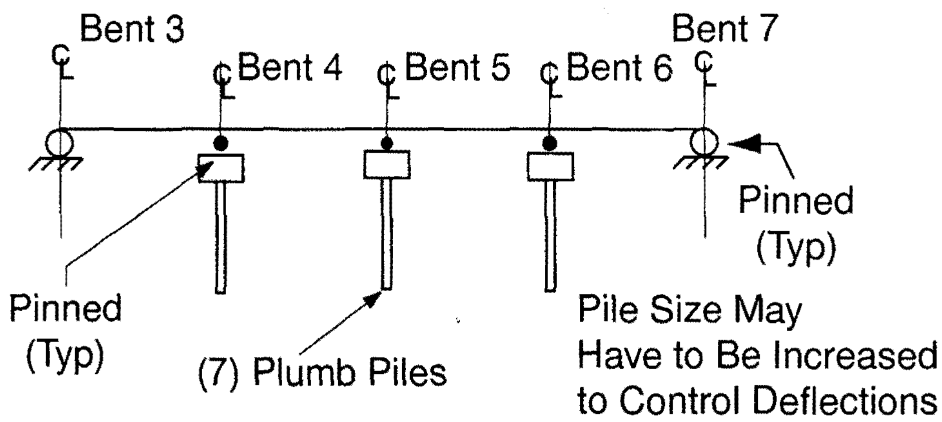
$$K_a = \frac{1}{\dfrac{1}{\dfrac{4.0(4030)144}{41.7}} + \dfrac{1}{2400(12)}} \sin^2(9.46°) = 513 \frac{\text{kip}}{\text{ft}}$$

$$K_a \sim 3.6\, K_f$$

Considerations for Batter Pile Designs (1 of 3)

- High Axial Stiffness Will Attract Large Seismic Forces
- In Some Cases, May Consider Using All Plumb Piles

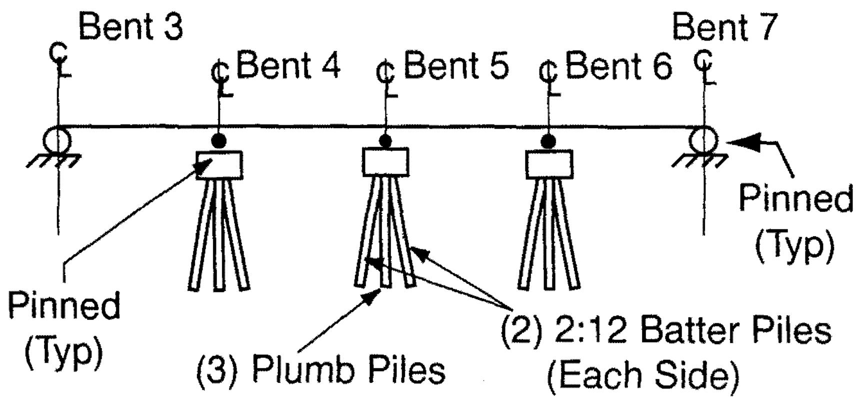
For Instance:



Considerations for Batter Pile Designs (2 of 3)

- More Than One 'Braced' Bent Per Frame May Be Required

For Instance:



R Factors for Pile Bents

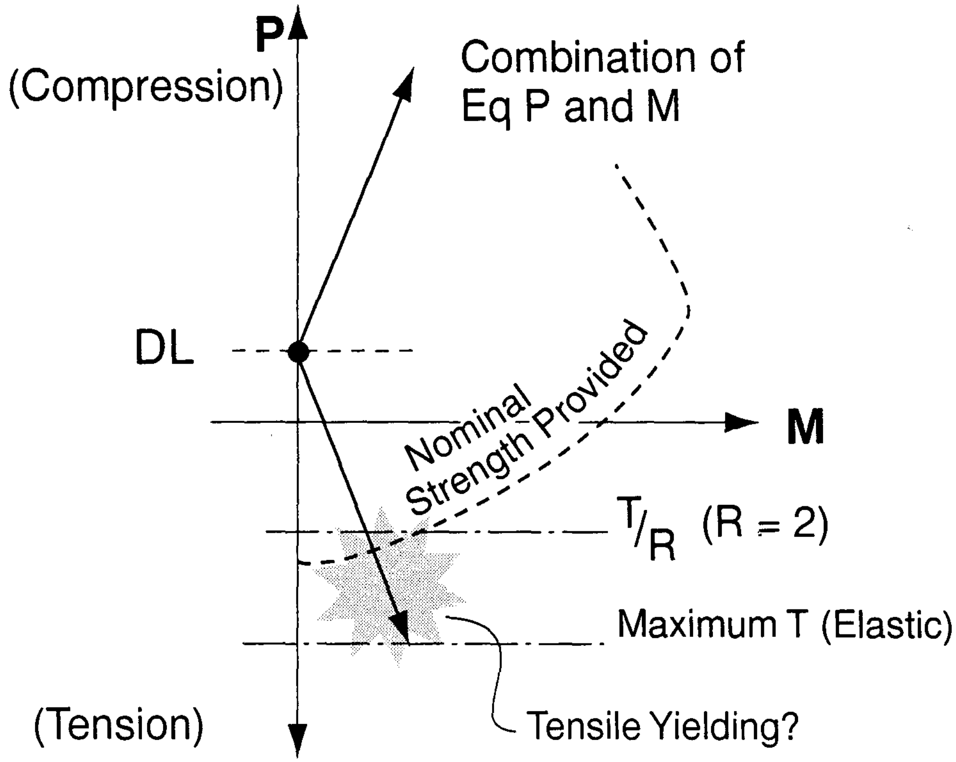
AASHTO Division IA, Table 3

	Concrete Piles	Steel Piles
All Piles Vertical (Plumb)	3	5
Some Piles Battered	2	3

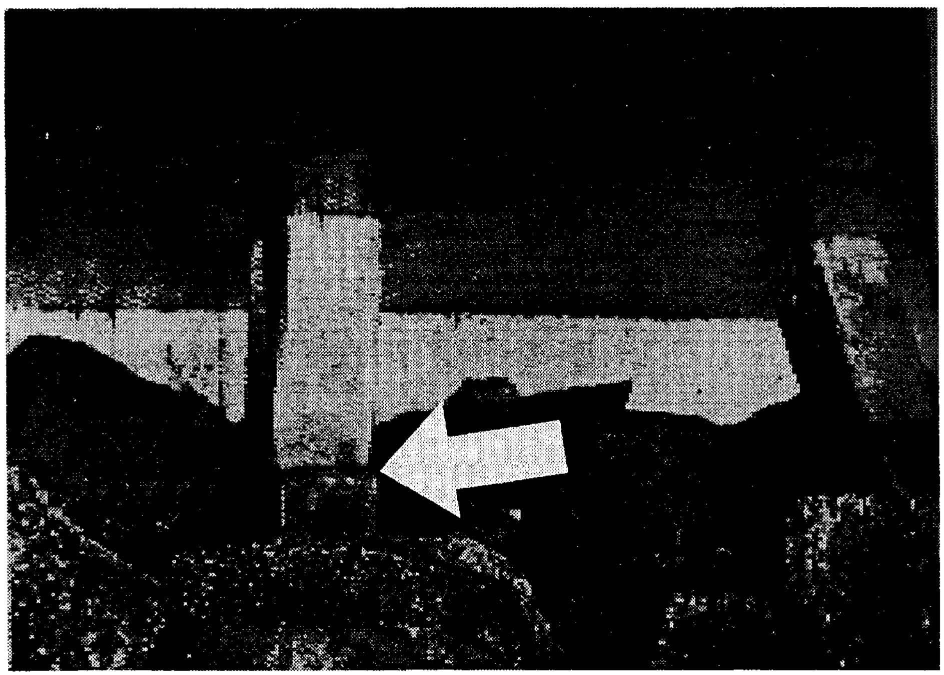
SPC B: Do Not Divide Above Factors by 2 for "Foundations"

SPC C and D: Use R = 1

Axial Force Issues



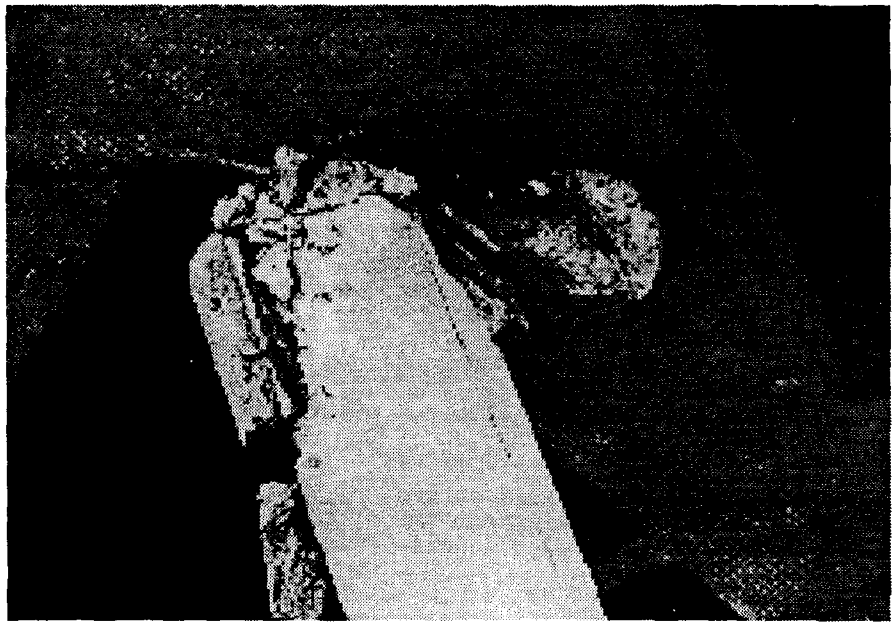
Consequences of Inadequate Tensile Strength / Batter Piles



Loma Prieta, 1989

EERI (1990)

Consequences of Inadequate Confinement / Plumb Piles



Loma Prieta, 1989

EERI (1990)

Considerations for Batter Pile Designs (3 of 3)

- Ductile Performance Is Associated with Plastic Hinging
- Axial Yielding Not Considered a Viable Ductile Mechanism
- Consider Designing with Elastic Forces?
  (At Least For Axial Forces in Pile)
- Large Axial Forces Transferred to Soil May Result in Residual Displacements
- Does Bridge Collapse? — Probably Not
- Is Bridge Serviceable After Earthquake? — Probably Not

Examples / Results for Center Frame of Bridge

Options:

1. One Bent with Batter Piles
2. All Plumb Piles
3. All Bents Have Battered Piles

Concrete Pile Options	Units	Longitudinal Direction			Transverse Direction
		Option 1	Option 2	Option 3	
Total Stiffness, K	kip/in.	587	258	1761	583
Period, T	sec	0.74	1.17	0.45	0.40
Total Seismic Shear, V	kip	550	447	845	225
Elastic Deflection, Δ	in	0.94	1.73	0.48	0.39
Max. Pile Tension	kip	-590		-238	
Max. Pile Compression	kip	846		494	
Max. Pile Moment, with R = 3	kip ft		340		192

Pile Tension Strength	kip	-213		-213	
Pile Compressive Strength	kip	767		767	
Pile Moment Strength	kip ft		370		370

Summary

- **Option No. 2** All Plumb Piles, Works Well
- **Option No. 3** Batter Piles in All Bents, Is Workable
- **Option No. 1** Batter Piles in One Bent, Does Not Work, too Much Load Is Attracted to too Few Batter Piles

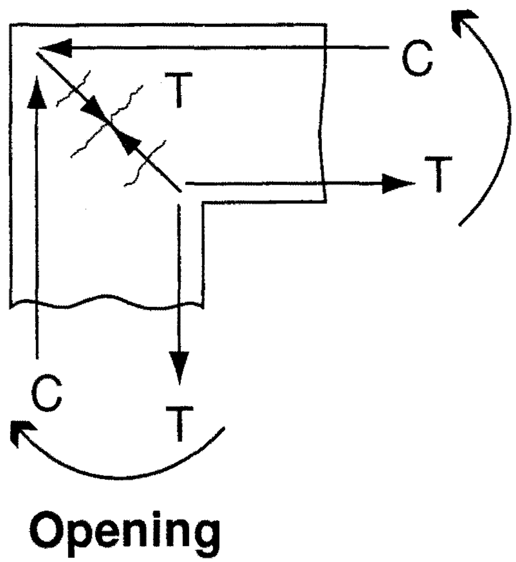
Conclusions

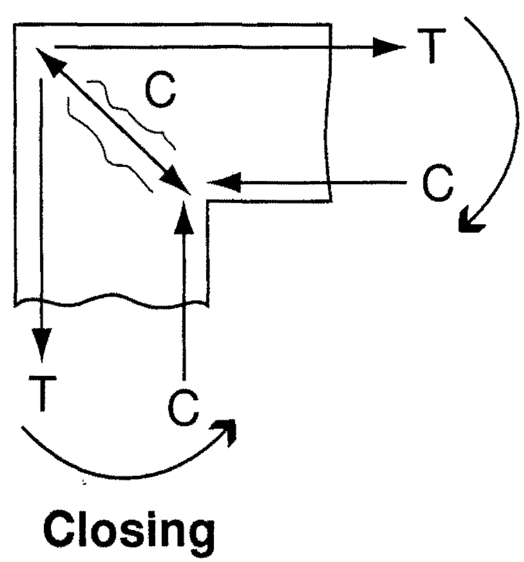
- Batter Piles Tend to Attract High Seismic Loads
- An All-Plumb Pile Solution May Be Better, Even if Pile Size Needs to Be Increased to Provide Adequate Stiffness
- If Batter Piles Are Used, Many Batter Piles May Be Necessary to Resist Seismic Loads

Session 6
Other Topics
Joint Design

- **Behavior**
- **Design Forces**
- **Shear Forces**

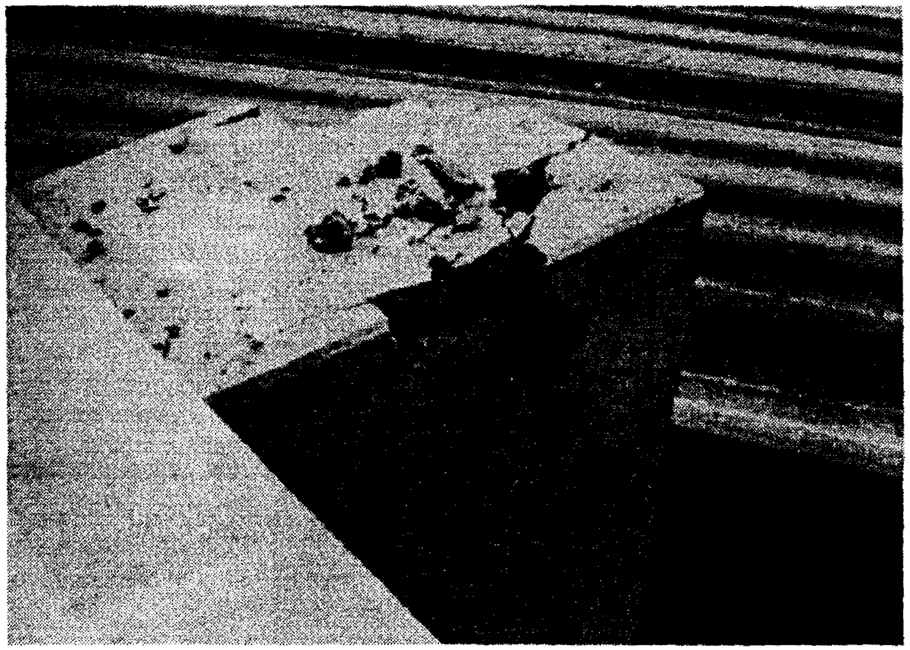
Behavior of Joints / Knee Joints





T = Tension
C = Compression

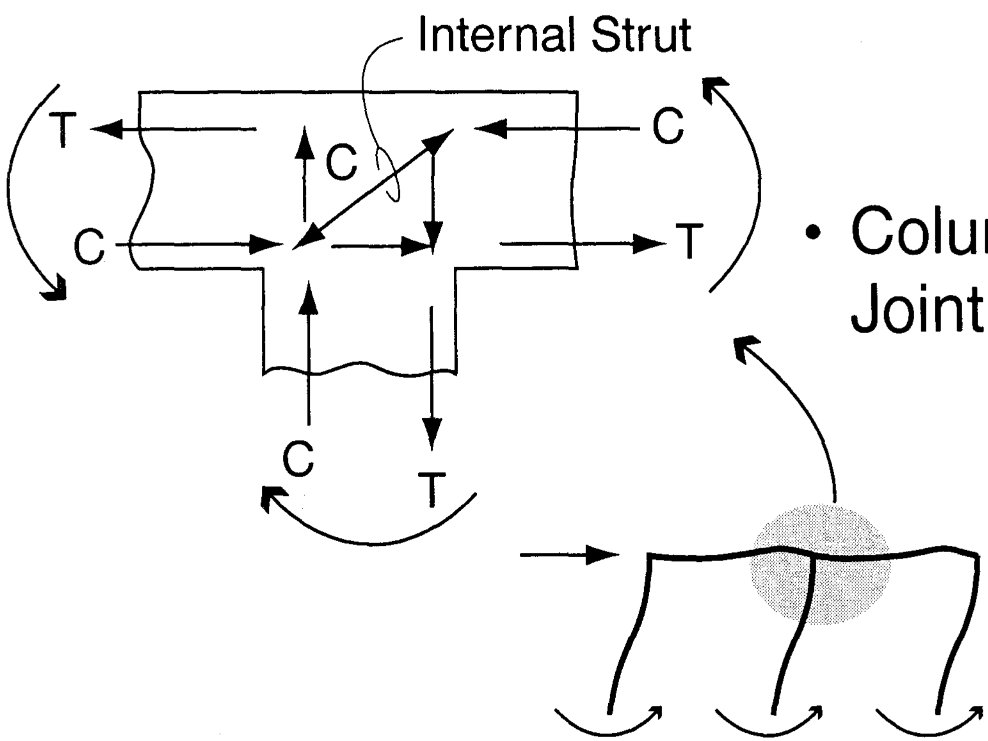
Knee Joint Damage



EERI (1990)

Loma Prieta, 1989

Behavior of Tee-Joints



- Column-to-Cap Beam Joint

Typical Frame

Design Practice

Empirical Joint Design Procedure

- Limit Magnitude of Average Joint Shear Stress
  (Limit Based on Experimental Data)

- Provide 'Minimum' Joint Confinement
  Steel Hoops to Preserve Integrity

Calculating Shear Forces

- **Option 1 Use Approximations**

$$v_j = \frac{M_p}{b_e\ h_b\ h_c}$$

Where

b_e = Effective Joint Width

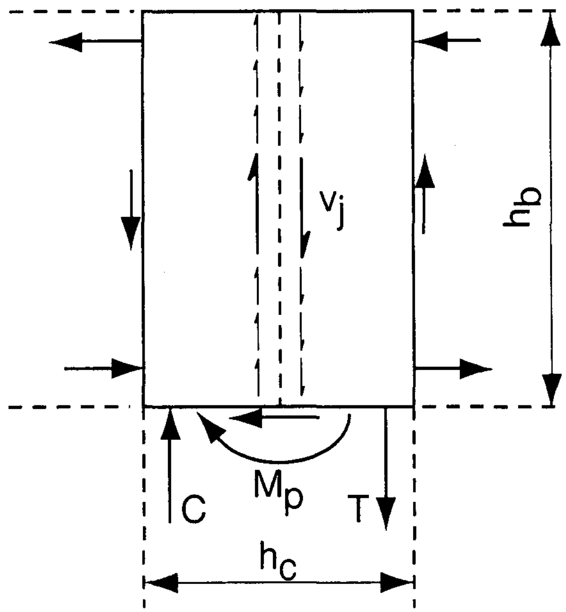
h_b = Beam Depth

h_c = Column Width

- **Option 2 Use Free Body Diagram with All Forces**

See Priestley, Seible, Calvi (1996)

Free Body of Joint



Approximations

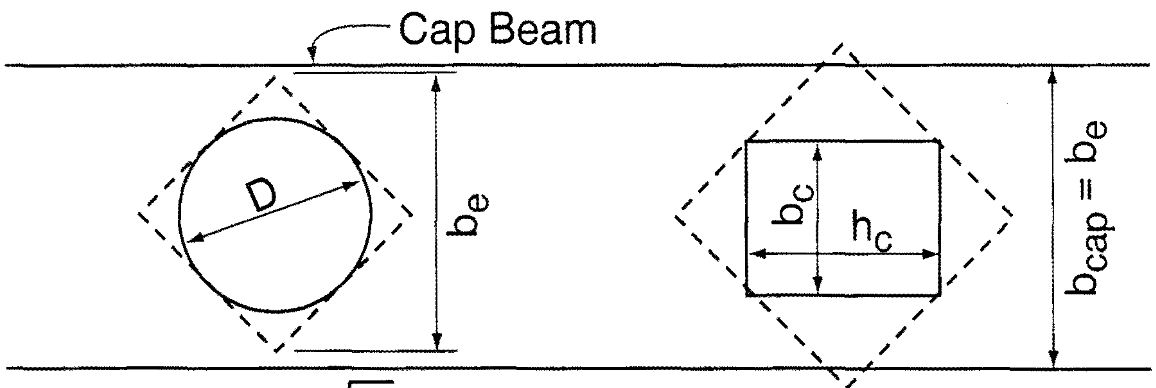
$$T \cong C \cong M_p/h_c$$

$$v_j \cong T/(b_e \bullet h_b)$$

$$v_j \cong \frac{M_p}{b_e\, h_b\, h_c}$$

Effective Joint Width

Circumscribe a Square About the Column



$b_e = D\sqrt{2}$

$b_e = b_c + h_c$

But: b_e not $> b_{cap}$

Plan View

Limiting Joint Shear Stress / Division I-A

- **SPC C and D** $v_j \leq 12\sqrt{f'_c}$ Normal Weight Concrete

  $v_j \leq 9\sqrt{f'_c}$ Light Weight Concrete

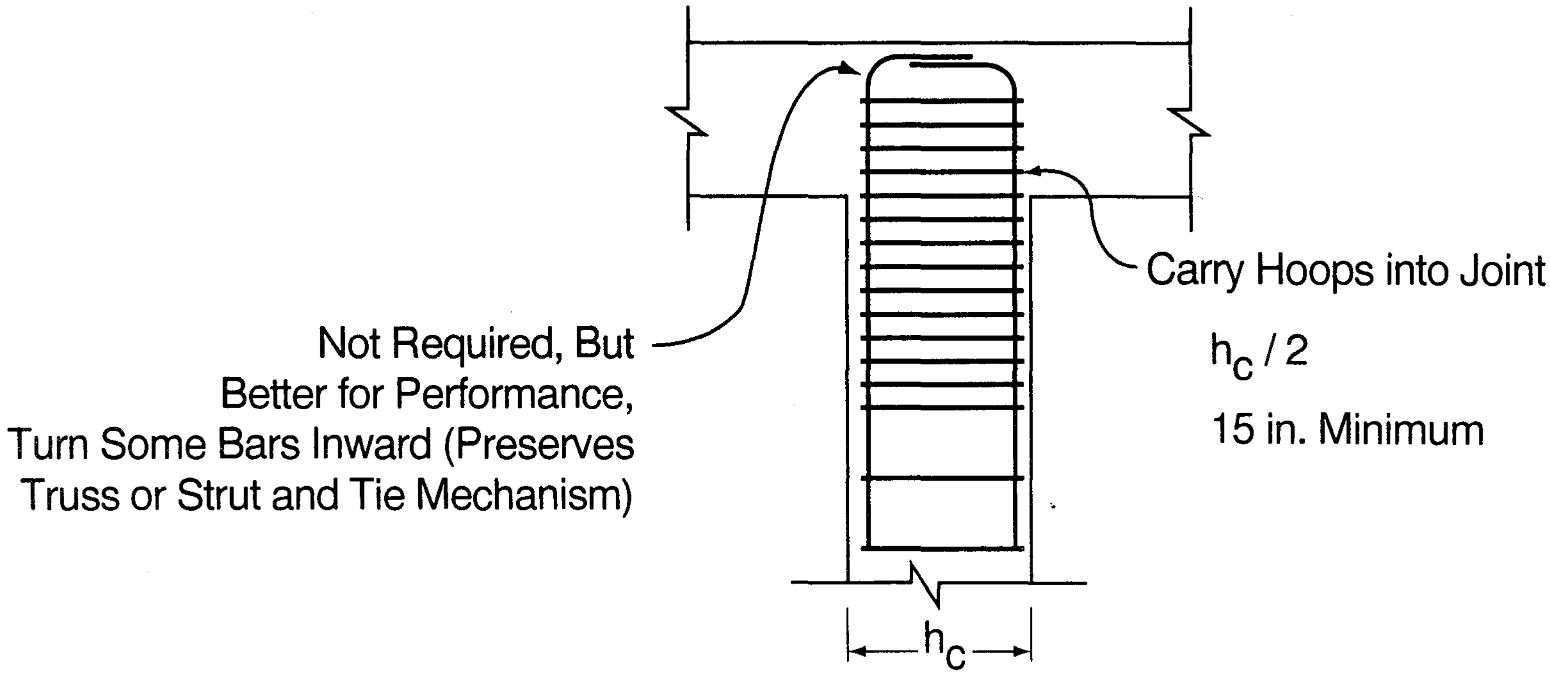
General Comments

- Current Method Provides No Increase Based on Amount of
  Confinement Steel, Which Is the Plastic-Hinge Confinement Steel Carried One-Half of Column Dimension into Adjoining Member, Not Less Than 15 in.

- If Stress Limit Not Met, Increase Cap Beam Size

- Other Methods in Development
  Truss Models
  Limiting Principal Tension in Joint

Detailing Considerations



Session 7
Other Topics
Existing Bridge Assessment and Retrofit

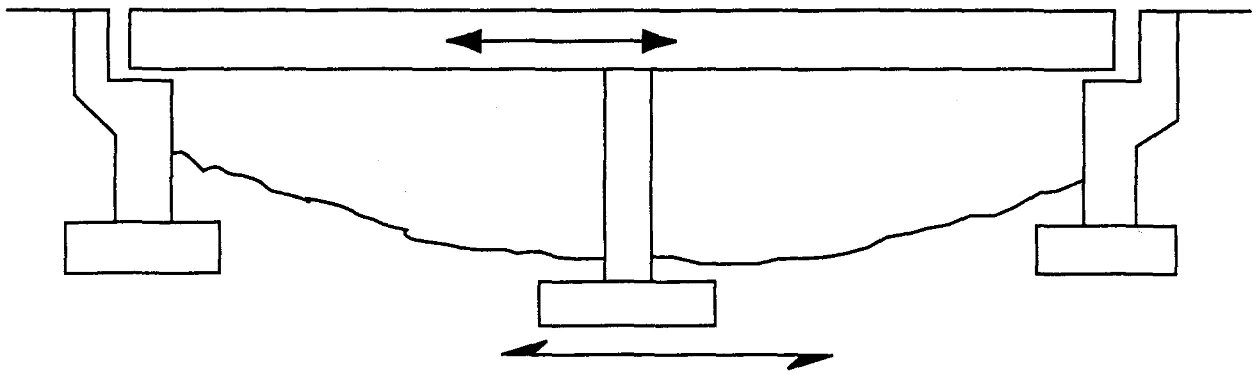
- Expected Performance
- Actual Behavior
- Assessment Methodologies
- Comparison of New Design and Retrofit Practice

Performance Objectives

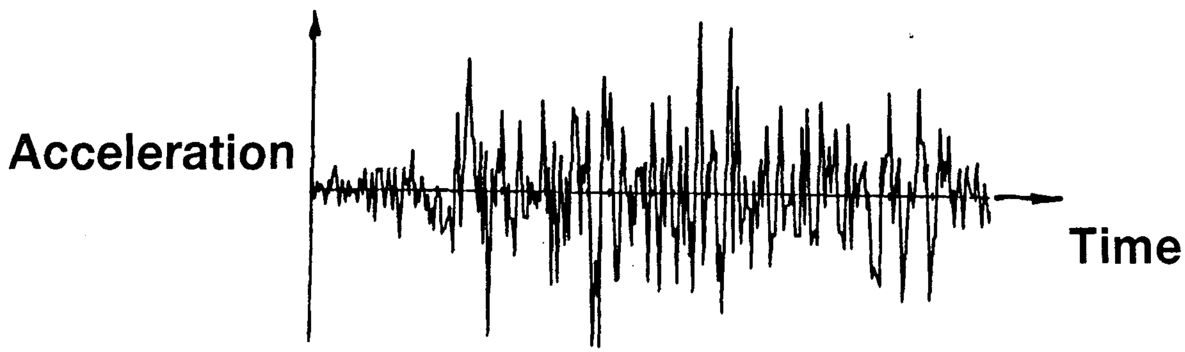
What Do We Expect from Our Bridges?

- Small to Moderate Earthquakes $\longrightarrow$ Elastic Response
  No Significant Damage
- Large, Infrequent Earthquakes $\longrightarrow$ Inelastic Response
  Damage Occurs, Detectable
  No Collapse

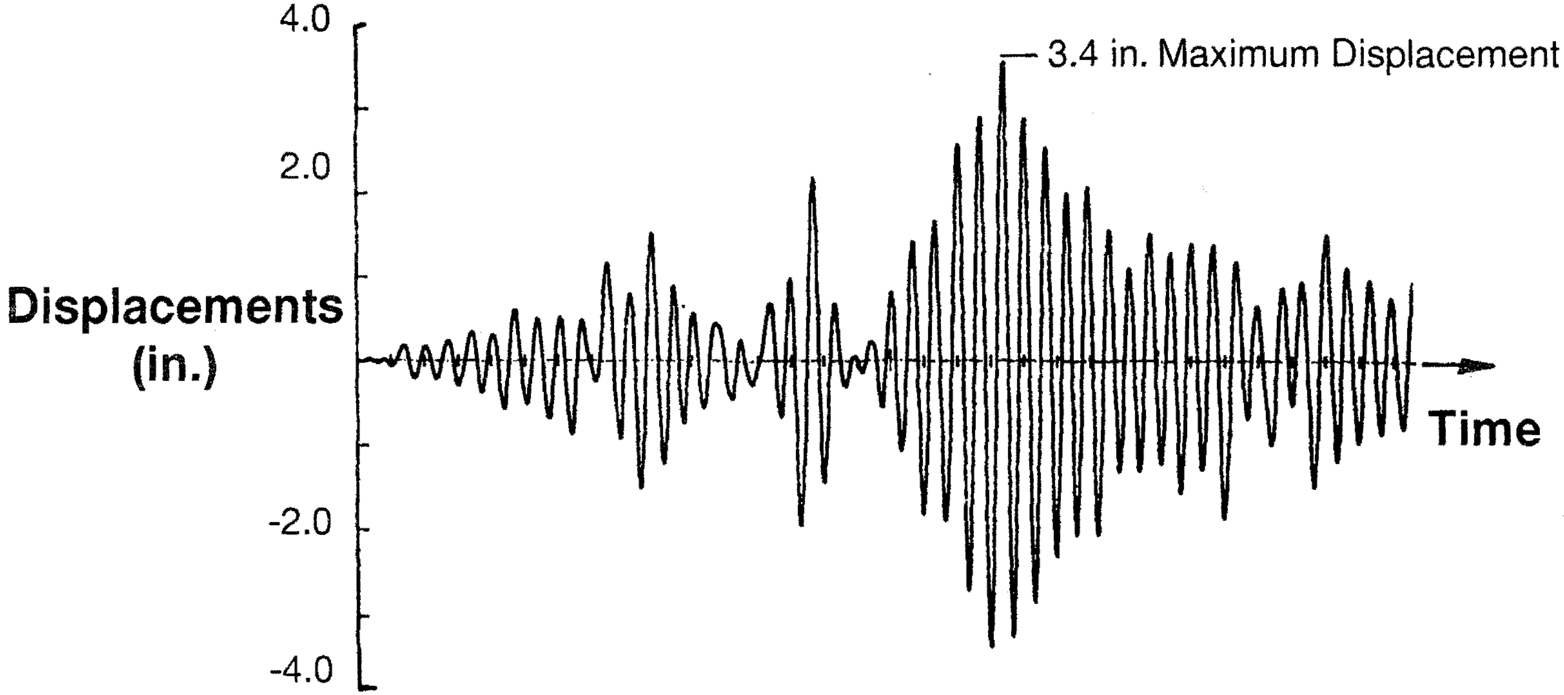
Conceptual Example



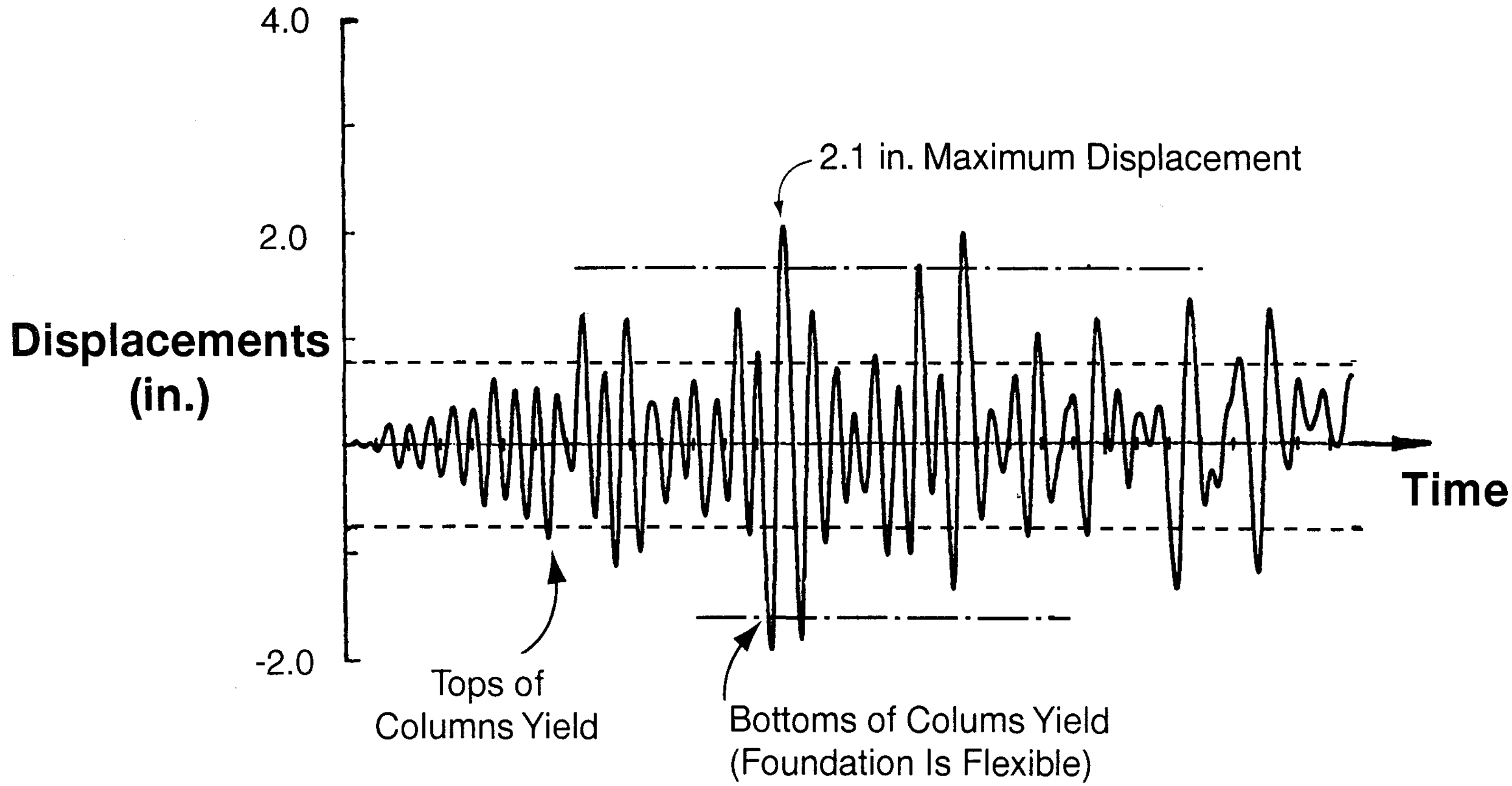
Longitudinal Earthquake Loading



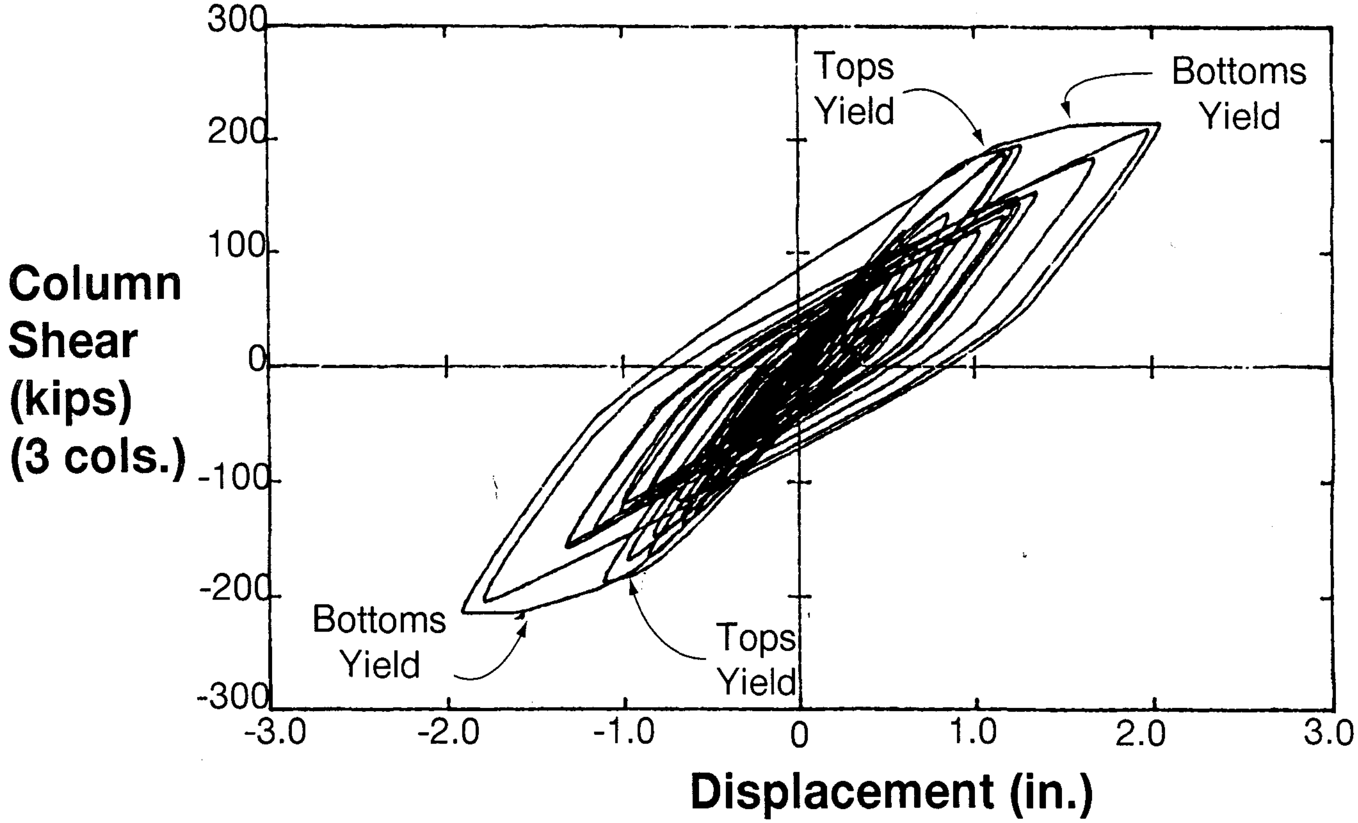
Response if Structure Remains Elastic



Response with Column Yielding



Shear in Column vs. Displacement



Performance of Example Bridge

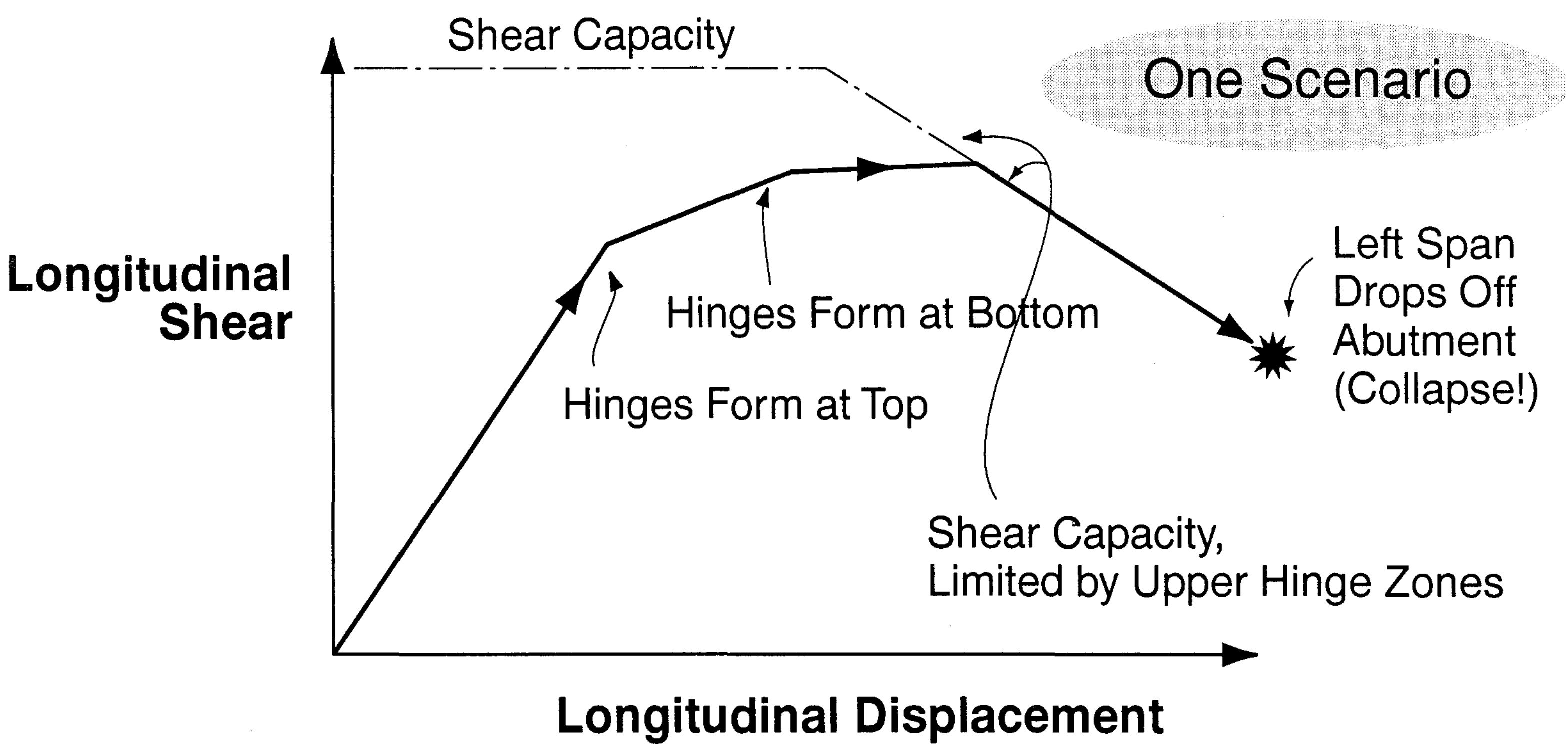
What Happened

- Yielding at Top of Columns
- Yielding at Bottom of Columns

What Did Not Happen

- Abutment Gap Did Not Close
- Footing Did Not Overturn
- Footing Soil OK
- Splice at Bottom OK
- No Shear Failure
  - Columns
    - Hinges
    - Middle
  - Joints

Quasi-Static Look at Behavior (Envelope)



Issues and Failure Modes to Consider

- Displacements at Abutments
- Displacements at Interior Expansion Joints
- Forces in Restrainers (If Present)
- Column Hinge Confinement (Plastic Hinge Rotation Capacity)
- Shear Strengths — Columns, Hinges, Footings, Joints, etc.
- Anchorage and Development / Splices
- Footing, Yielding, Overturning, Sliding
- Foundation Strength / Liquefaction

Assessment Methodologies

- Capacity / Demand Ratio Method
- Lateral Strength Method (FHWA)

  [Plastic Collapse Mechanism
  Pushover]

Capacity / Demand Ratio Method (1 of 3)

- Analyze Bridge Elastically to Obtain Demands
- Calculate Member / Item Capacities

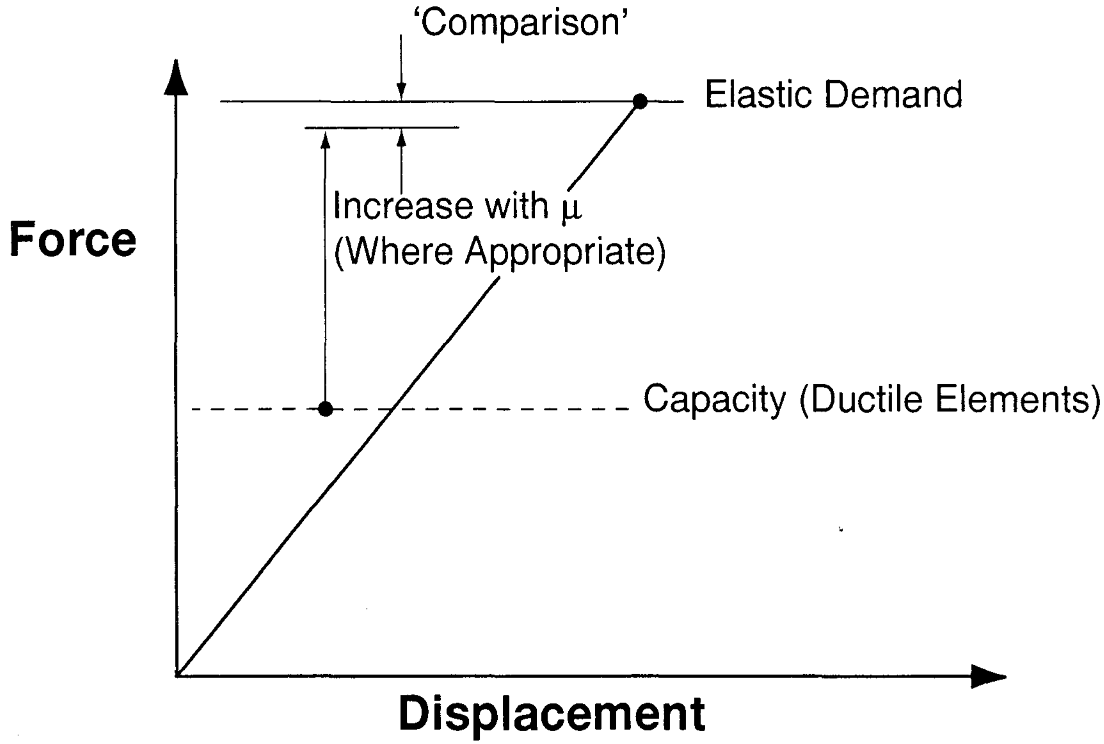
  (ϕ = 1.0, Nominal Ultimate Values)
- Form C/D Ratio
- Increase Ratios for Ductile Elements

  Using Ductility Indicator, μ

  $$\frac{C}{D} < 1.0 \longrightarrow \text{Failure}$$
- Estimate Damage / Failure Likelihoods

  (Lowest C/D First, etc.)

Capacity / Demand Ratio Method (2 of 3)



Capacity / Demand Ratio Method (3 of 3)

Advantages

- Simple Analysis
- Quick Ranking of Element Performance
- Relatively Comprehensive Comparisons Developed

Disadvantages

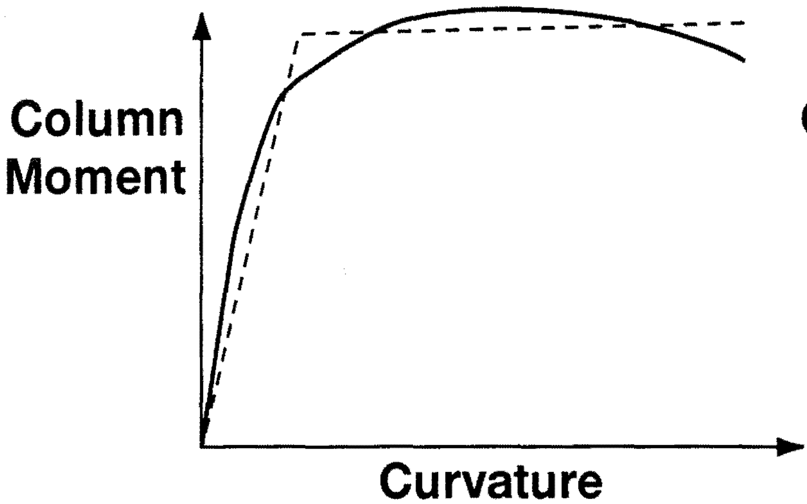
- Focus Is Entirely on Element Performance
- Cannot Account for Force Redistribution
- Does Not Account for Capacity Protection of Elements

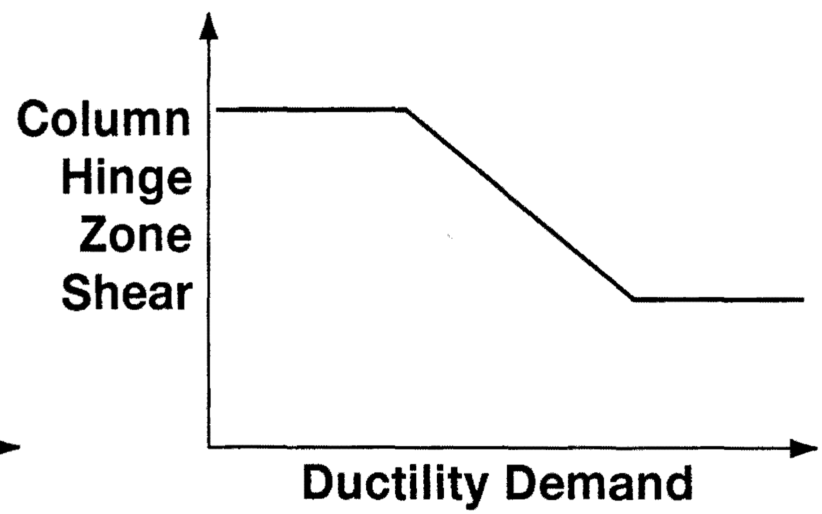
Lateral Strength (Pushover) Method (1 of 4)

- Analyze Bridge Elastically to Obtain Target Displacements
- Develop Member Yield / Deformation / Failure Relations
- Develop Static Force / Resistance Curves (Pushover)
    - Entire Structure
    - Individual Frames
- Evaluate Behavior Up to Target Displacement

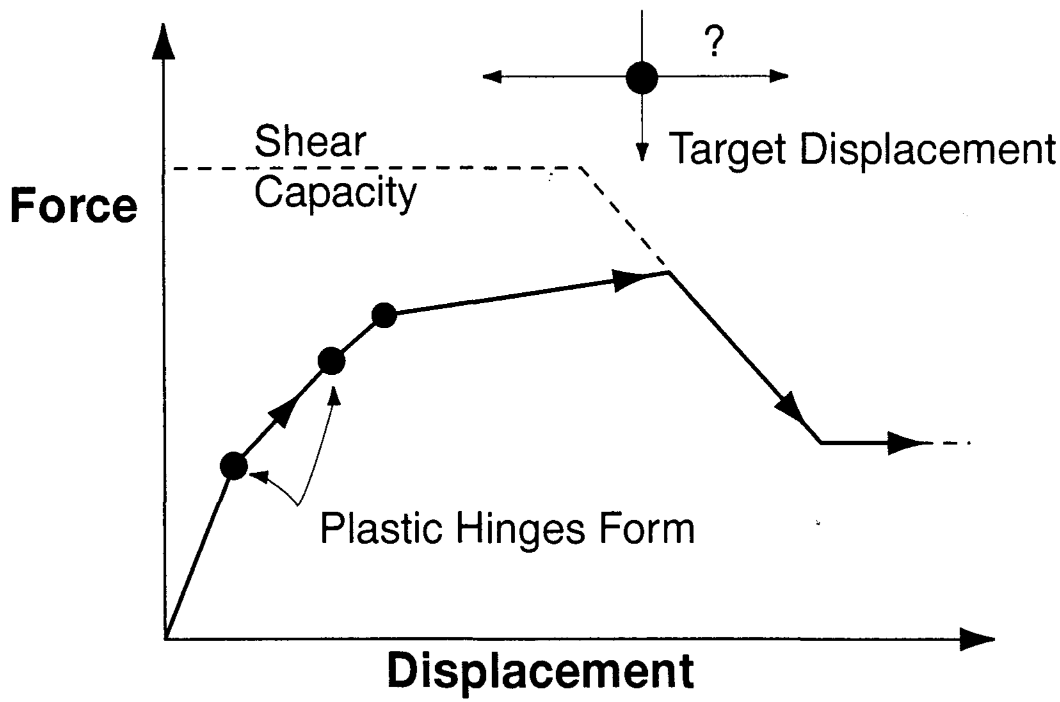
Can Elements Endure Entire Displacement Sequence?

Lateral Strength (Pushover) Method (2 of 4)





Lateral Strength (Pushover) Method (3 of 4)



Lateral Strength (Pushover) Method (4 of 4)

Advantages

- Tracks Sequence of Events (Yielding, Degradation, etc.) in Structure
- Indicates Structure (Sub-Structure) Overall Response — **System Focus**

Disadvantages

- More Effort Required (Development of Basic Member Data)
- Does Not Address Cyclic Effects Directly

New Design vs. Assessment / Retrofit

Item	New Design Provisions	Existing Bridges
• **Plastic Hinging**	Prescriptive Confinement	Assess Rotation Capacity Add Jacketing
• **Member Shear**	Design for Plastic Hinging Forces	Assess Shear Capacity and Ductility Demand Add Jacketing
• **Structure Displacements**	Provide Wide Seats	Probable Displacements Extend Seats Add Restrainers

New Design vs. Assessment / Retrofit

Item	New Design Provisions	Existing Bridges
• **Reinforcement Splices**	No Splices in High Moment Zone	Assess Ductility Demand Add Jacketing
• **Footing Yielding** **Footing Shear**	Design for Plastic Add Overlay Enlarge Footing	Assess Probable Forces Hinging Forces
• **Joint Shear**	Limit Average Shear Stress Protect from Force	Enlarge Joint Add Jacketing

Seismic Bridge Design Applications
Concluding Considerations

- In the Wake of the **1994 Northridge Earthquake:**

  The Seismic Advisory Board Appointed to Evaluate Caltrans' Efforts Concluded:

  > **"Caltrans' design procedures and retrofit procedures are 'technically sound."**

$$\frac{\text{7 Collapses}}{\sim \text{4823 Total Bridges in LA Co.}}$$

- Caltrans' efforts → AASHTO Division I-A

  Other's Experience (NZ, Japan, etc.) → AASHTO Division I-A

Long Way from 1971!

Seismic Bridge Design Applications

Questions and Answers

Selected References

- **BOOKS**

Berg, G.V. (1989), *Elements of Structural Dynamics*, Prentice-Hall.
Bolt, B.A. (1988), *Earthquakes*, Freeman.
Bowles, J.E. (1988), *Foundation Analysis and Design*, McGraw-Hill.
Clough, R.W. & Penzien, J. (1975/1993), *Dynamics of Structures*, McGraw-Hill.
Dowrick, D.J. (1977/1987), *Earthquake Resistant Design: A Manual for Engineers and Architects*, Wiley.
Naeim, F. (editor) (1989), *The Seismic Design Handbook*, Van Nostrand Reinhold.
Newmark, N.M. & Rosenbleuth, E. (1971), *Fundamentals of Earthquake Engineering*, Prentice-Hall.
Park, R. & Paulay, T. (1975), *Reinforced Concrete Structures*, Wiley.
Paulay, T. & Priestley, M.J.N. (1992), *Seismic Design of Reinforced Concrete and Masonry Buildings*, Wiley.
Peck, R.B., Hanson, W.A., and Thornburn, T.H. (1974), *Foundation Engineering*, Wiley.
Poulos, H.G. and Davis, E.H. (1980), *Pile Foundation Analysis and Design*, Wiley.
Priestley, M.J.N., Seible F., Calvi, G.M. (1996), *Seismic Design and Retrofit of Bridges*, Wiley.
Reiter, L. (1990), Earthquake Hazard Analysis, *Issues and Insights*, Columbia University Press.
Scott, R.F. (1981), *Foundation Analysis*, Prentice-Hall.
Wakabayashi, M. (1986), *Design of Earthquake Resistant Buildings*, McGraw-Hill.

Selected References (continued)

- **AASHTO DOCUMENTS**

  Standard Specifications for Highway Bridges, Interim Specifications — 1995.
  Guide Specifications for Seismic Isolation Design — 1991.

- **STATE DEPARTMENT OF TRANSPORTATION MANUALS**

  State of California, Department of Transportation

  Caltrans (1995), *Bridge Design Specifications*, Sacramento, CA.
  Caltrans (1989), *Bridge Design Aids Manual*, Sacramento, CA.
  Caltrans (1986), *Bridge Memo to Designers*, Sacramento, CA.
  Caltrans (1986), *Bridge Design Details Manual*, Sacramento, CA.
  Caltrans (1983), *Bridge Design Practice Manual*, Sacramento, CA.

  Washington State Department of Transportation

  WSDOT (1995), *Bridge Design Manual*, Olympia, WA.

Selected References (continued)

- **EARTHQUAKE RECONNAISSANCE REPORTS**

Caltech (1971), *Engineering Features of the San Fernando Earthquake*, EERL 71 - 02, California Institute of Technology.

EERI (1995), *Northridge Earthquake Reconnaissance Report, Vol. 1,* Earthquake Engineering Research Institute.

EERI (1990), *Loma Prieta Earthquake Reconnaissance Report,* Earthquake Engineering Research Institute.

UCSD (1994), *The Northridge Earthquake of January 17, 1994, Damage Analysis of Selected Freeway Bridges* SSRP-94/06, University of California, San Diego.

- **OTHER**

NAVFAC (1986), *Foundations and Earth Structures, Design Manual 7.02,* Naval Facilities Engineering Command, Alexandria, VA.

Roeder and Stanton (1990), Design of Laminated Elastomeric Bridge Bearings, *Transportation Research Record,* 1290.

Seed and Idriss (1970), *Soil Moduli and Damping Factors for Dynamic Response Analyses,* EERC 70-10, University of California, Berkeley.

Selected References (continued)

- **EERI MONOGRAPH SERIES (Earthquake Engineering Research Institute, Oakland, CA)**

  Algermissen, S.T. (1983), *An Introduction to the Seismicity of the United States.*
  Berg, G.V. (1983), *Seismic Design Codes and Procedures.*
  Chopra, A.K. (1981), *Dynamics of Structures, A Primer.*
  Hudson, D.E. (1979), *Reading and Interpreting Strong Motion Accelerograms.*
  Housner, G.W. and Jennings, P.C. (1982), *Earthquake Design Criteria.*
  Newmark, N.M. and Hall, W.J. (1982), *Earthquake Spectra and Design.*
  Seed, H.B. and Idriss, I.M. (1983), *Ground Motions and Liquefaction During Earthquakes.*

- **FEDERAL HIGHWAY ADMINISTRATION (National Technical Information Service, Springfield, VA)**

  FHWA-IP-81-2 (1981), *Seismic Design of Highway Bridges - Workshop Manual.*
  FHWA-IP-87-6 (1987), *Seismic Design and Retrofit Manual for Highway Bridges.*
  FHWA-HI-88-042 (1988), *Drilled Shafts.*
  FHWA-RD-86-101, 102, 103 (1986), *Seismic Design of Highway Bridge Foundations.*