Structural Systems
Concrete Systems
One-Way vs Two-Way systems
Flat plate systems
Beam and joist systems
PT vs Mild Reinforcing
Tilt Panels
Precast
Steel Systems
Composite Framing
Steel Joists
Steel deck
Metal Buildings
Bolts vs Welds
Wood Systems

Post and Beam vs Bearing Wall
Solid Sawn Lumber
Composite/Manufactured Lumber
Glulam Beams & Columns
Open web trusses (Floor and Roof)
Heavy Timber
Mass timber
CFS Systems
Stud & Joist sizes
Trusses
Deck & Plank
Load bearing vs non-load bearing
Proprietary Systems
Questions?
1. $R_A = ?$
2. $R_B = ?$
BEAM ANALYSIS

Simplify Loads.

Rules to Remember:
• \( \Sigma F = 0 \)
• \( \Sigma M \) about one point = 0
BEAM ANALYSIS

$\Sigma M$ about one point = 0

$\Sigma M_A = 0$

$\Sigma M_A = (50 \text{ kips} \times 15 \text{ ft}) + (R_B \times 20 \text{ ft}) = 0$

$\Sigma M_A = 750 \text{ kips} + 20R_B = 0$

$R_B = -750 \text{ kips} / 20$

$R_B = 37.5 \text{ kips}$
BEAM ANALYSIS

\[ \Sigma F = 0 \]

\[ R_A - 50 \text{ kips} + 37.5 \text{ kips} = 0 \]

\[ R_A = 12.5 \text{ kips} \]
COLUMN ANALYSIS

Floor Dead Load = 35 psf
Floor Live Load = 40 psf
HSS6x6x1/4 columns; r = 2.34 in

1. What is the center column load?
2. What is the maximum allowable height of the column, considering stability?
COLUMN ANALYSIS

Determine Loads.

Floor Dead Load = 35 psf
Floor Live Load = 40 psf

Total Factored Area Load =
(1.2 x 35 psf) + (1.6 x 40 psf) =
106 psf
COLUMN ANALYSIS

Determine tributary area

\[ A_T = \left[ \frac{(20 \text{ ft} + 30 \text{ ft})}{2} \right] \times \left[ \frac{(20 \text{ ft} + 20 \text{ ft})}{2} \right] \]

\[ A_T = 500 \text{ sqft} \]
COLUMN ANALYSIS

Determine column load

\[ P = A_T \times \text{area load} = 106 \text{ psf} \times 500 \text{ sqft} \]

\[ P = 53 \text{ kips} \]
COLUMN ANALYSIS

Determine maximum column length for stability

For a steel column, this limit is as follows:

\[ KL/r < 200 \]

- **K** = Effective Length Factor
- **L** = Unbraced Length of Column
- **r** = Radius of Gyration

**TABLE C-A-7.1**
Approximate Values of Effective Length Factor, **K**

<table>
<thead>
<tr>
<th>End condition code</th>
<th>Theoretical <strong>K</strong> value</th>
<th>Recommended design value when ideal conditions are approximated</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rotation free and translation free</td>
<td>0.65</td>
<td>0.65</td>
</tr>
<tr>
<td>Rotation fixed and translation free</td>
<td>0.80</td>
<td>0.80</td>
</tr>
<tr>
<td>Rotation free and translation fixed</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Rotation fixed and translation free</td>
<td>1.2</td>
<td>1.2</td>
</tr>
<tr>
<td>Rotation free and translation free</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Rotation fixed and translation free</td>
<td>2.0</td>
<td>2.0</td>
</tr>
</tbody>
</table>
COLUMN ANALYSIS

KL/r < 200

r = 2.34 in
COLUMN ANALYSIS

Determine maximum column length for stability

For a steel column, this limit is as follows:

\[ KL/r < 200 \]

- **K** = Effective Length Factor
- **L** = Unbraced Length of Column
- **r** = Radius of Gyration
COLUMN ANALYSIS

KL/r < 200

r = 2.34 in
K = 0.7

0.7 x L / 2.34 = 200
L = 200 x 2.34 / 0.7 = 668 in
668 in / 12 in/ft = 55 ft maximum
Questions?
FOUNDATIONS
SHALLOW FOUNDATIONS

COLUMN FOOTING

WALL FOOTING

COMBINED FOOTING

MAT FOOTING

STRAP FOOTING
Shallow Foundation Design

Shallow foundations are types of foundations that are supported from the soil. Typically used on lighter and shorter buildings. Bearing pressures typically vary from 2,000 psf to 7,000 psf. Typical to consider uniform distribution of load across full area of footing. Shape and proportion of footings can impact bearing capacity.
TYPICAL SPREAD FOOTING AT COLUMN
Deep Foundation Design

Deep foundations are defined as foundations whose depth is larger than its width.

Typically used for higher capacities and reduced settlement.

Simple terms is to consider installing columns into the ground.

Capacities are distributed vertically rather than horizontally.

Can rely on both skin friction and end bearing.
DRILLED PIER FOUNDATIONS

(a) (b) (c) (d)

STRAIGHT SHAFT BELLED PIER

Roughened or grooved sidewall to transmit shear

Fill
Soil
Soft to sound rock
Shear support
End bearing

Poor bearing soil
Good bearing soil
End bearing

End bearing
DRILLED PIERS

Typically use larger diameters and single piers vs grouped to increase capacity

Steel casing may be required if soils are prone to caving or if the water table is present

Casings will reduce skin friction resistance capacity

Caisson is just a drilled pier with full water-tight casing.

Reinforcing cages treated similar to a concrete column but do not necessarily need to go full depth
PILE FOUNDATIONS

(a) Group and single pile on rock or very firm soil stratum.

(b) Group or single pile “floating” in soil mass.

(c) Offshore pile group.

(d) Tension pile.

(e) Pile penetrating below a soil layer that swells (shown) or consolidates.
Many types of Piles:
Augercast / CIP piles
Precast piles
Steel piles
Wood Piles
Sheet Piles
Micro/Macro Piles
Driven vs Drilled
RETAINING WALLS
Types of Retaining Walls
Cantilevered Retaining Walls Modes of Failure
RETAINING WALL LOADING

**DESIGN FORCES**
- \( V = W_s \times L_{ef} + W_s + W_w \)
- \( H = H_s + H_a \)
- \( M_{ef} = H_a \times H/2 + H_s \times H/3 \)

**DESIGN RESISTANCE**
- \( Q_{max} = V/L_v + 6 \times M_{ef} / L_v^2 \leq q_{S,max} \)
- \( H_s = H_a + F \geq 1.5H \)
- \( M_s = W_s \times (L_v/2 + L_{no} + L_r) + W_w \times (L_r + L_v/2) \geq 2 \times M_{ef} \)
- \( e = M_{ef} / V \leq L_v / 6 \) - to avoid back of footing in tension
Questions?
Retaining Wall Practice Problems
RETAINING WALL PROBLEMS

A. Determine the horizontal shear force acting on the wall

B. Determine overturning moment.

C. Does the wall require a shear key?

GIVENS
Active Earth Pressure = 35 psf/ft
Passive Earth Pressure = 375 psf/ft
Soil Density = 120 pcf
Concrete Density = 150 pcf
Coefficient of Friction = 0.35
Axial Dead Load = 20 plf psf
Surcharge Lateral Load = 150 psf
A. Horizontal Shear

\[ H_{\text{soil}} = 35 \times 10 \times 10 / 2 = 1,750 \text{ lbs/ft} \]

\[ H_{\text{surcharge}} = 150 \times 8.5 = 1,275 \text{ lbs/ft} \]

\[ H_{\text{total}} = H_{\text{soil}} + H_{\text{surcharge}} \]

\[ H_{\text{total}} = 1,750 + 1,275 = 3,025 \text{ lbs/ft} \]
RETAINING WALL PROBLEMS

B. Overturning Moment

\( M_{\text{soil}} = 1,750 \times 3.33 = 5,833 \text{ lb-ft/ft} \)

\( M_{\text{surcharge}} = 1,275 \times 5.75 \)

= 7,331 lbs-ft/ft

\( M_{\text{total}} = M_{\text{soil}} + M_{\text{surcharge}} \)

\( M_{\text{total}} = 5,833 + 7,331 \)

= 13,165lbs-ft/ft
C. Sliding

\[ H_P = 375 \times 2.5 \times 2.5/2 = 1,172 \text{ lb/ft} \]

\[ W_{\text{soil}} = 120 \times 8.5 \times 4 = 4,080 \text{ lb/ft} \]

\[ W_{\text{wall}} = 150 \times 1 \times 6 = 900 \text{ lb/ft} \]

\[ W_{\text{fg}} = 150 \times 1.5 \times 7.25 = 1,631 \text{ lb/ft} \]

\[ W_{\text{total}} = 4,080 + 900 + 1,631 = 6,611 \text{ lb/ft} \]

\[ H_{DL} = 0.35 \times 6,611 = 2,314 \text{ lb/ft} \]

\[ H_{\text{resisting}} = 1,172 + 2,314 = 3,486 \text{ lb/ft} \]

\[ H_{\text{resisting}}/H_{\text{total}} = 3,486 / 3,025 \text{ lb/ft} = 1.15 \]

1.15 < 1.5 – NEED SHEAR KEY
Questions?
LATERAL FORCES

ROLAND HILL, P.E.
Wind and Seismic Forces to Buildings

Wind Base Shear

\[ P = Q_s \cdot (G_{Cp} \pm G_{cp}) \cdot A \]

Seismic Base Shear

\[ V = C_s \cdot W = \Sigma F \]
DETERMINING WIND FORCES
Main Wind Force Resisting System (MWFRS) Forces vs Components and Cladding (C&C) Forces

USE DIFFERENT MAGNITUDE FORCES FOR DESIGN
Wind Design (MWFRS) per ASCE 7

\[ P_w = Q_s \times (GCp \pm Gcpi) \times A \]

\[ Q_s = \text{Wind Velocity Pressure} \]

\[ Q_s = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \]

\[ V = \text{Basic Wind Speed (mph)} \]

\[ K \text{ factors = Height, Topography & Direction} \]

\[ GCp = \text{Product of Gust Effect and Wind Pressure coefficients for external and internal pressures} \]

\[ A = \text{Surface Area} \]
Basic Wind Speed Map
V (mph)
Maps based on:
Location and Building Risk Classification
### K Factors Tables

- **Kz – Height Factor**
  - Varies on height and exposure

- **Kd – Directional Factor**
  - Varies on structure type

- **Kzt – Topographic Factor**

#### Kz – Height Factor

<table>
<thead>
<tr>
<th>Height above ground level, $z$</th>
<th>$B$</th>
<th>$C$</th>
<th>$D$</th>
</tr>
</thead>
<tbody>
<tr>
<td>ft (m)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0-15 (0-4.6)</td>
<td>0.57</td>
<td>0.85</td>
<td>1.03</td>
</tr>
<tr>
<td>20 (6.1)</td>
<td>0.62</td>
<td>0.90</td>
<td>1.08</td>
</tr>
<tr>
<td>25 (7.6)</td>
<td>0.66</td>
<td>0.94</td>
<td>1.12</td>
</tr>
<tr>
<td>30 (9.1)</td>
<td>0.70</td>
<td>0.98</td>
<td>1.16</td>
</tr>
<tr>
<td>40 (12.2)</td>
<td>0.76</td>
<td>1.04</td>
<td>1.22</td>
</tr>
<tr>
<td>50 (15.2)</td>
<td>0.81</td>
<td>1.09</td>
<td>1.27</td>
</tr>
<tr>
<td>60 (18)</td>
<td>0.85</td>
<td>1.13</td>
<td>1.31</td>
</tr>
<tr>
<td>70 (21.3)</td>
<td>0.89</td>
<td>1.17</td>
<td>1.34</td>
</tr>
<tr>
<td>80 (24.4)</td>
<td>0.93</td>
<td>1.21</td>
<td>1.38</td>
</tr>
<tr>
<td>90 (27.4)</td>
<td>0.96</td>
<td>1.24</td>
<td>1.40</td>
</tr>
<tr>
<td>100 (30.5)</td>
<td>0.99</td>
<td>1.26</td>
<td>1.43</td>
</tr>
<tr>
<td>120 (36.6)</td>
<td>1.04</td>
<td>1.31</td>
<td>1.48</td>
</tr>
<tr>
<td>140 (42.7)</td>
<td>1.09</td>
<td>1.36</td>
<td>1.52</td>
</tr>
<tr>
<td>160 (48.8)</td>
<td>1.13</td>
<td>1.39</td>
<td>1.55</td>
</tr>
<tr>
<td>180 (54.9)</td>
<td>1.17</td>
<td>1.43</td>
<td>1.58</td>
</tr>
<tr>
<td>200 (61.0)</td>
<td>1.20</td>
<td>1.46</td>
<td>1.61</td>
</tr>
<tr>
<td>250 (76.2)</td>
<td>1.28</td>
<td>1.53</td>
<td>1.68</td>
</tr>
<tr>
<td>300 (91.4)</td>
<td>1.35</td>
<td>1.59</td>
<td>1.73</td>
</tr>
<tr>
<td>350 (106.7)</td>
<td>1.41</td>
<td>1.64</td>
<td>1.78</td>
</tr>
<tr>
<td>400 (121.9)</td>
<td>1.47</td>
<td>1.69</td>
<td>1.82</td>
</tr>
<tr>
<td>450 (137.2)</td>
<td>1.52</td>
<td>1.73</td>
<td>1.86</td>
</tr>
<tr>
<td>500 (152.4)</td>
<td>1.56</td>
<td>1.77</td>
<td>1.89</td>
</tr>
</tbody>
</table>

#### Kd – Directional Factor

<table>
<thead>
<tr>
<th>Structure Type</th>
<th>Directionality Factor $K_d$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arched Roofs</td>
<td>0.85</td>
</tr>
<tr>
<td>Chimneys, Tanks, and Similar Structures</td>
<td>0.90</td>
</tr>
<tr>
<td>Solid Freestanding Walls and Solid Freestanding and Attached Signs</td>
<td>0.85</td>
</tr>
<tr>
<td>Open Signs and Lattice Framework</td>
<td>0.85</td>
</tr>
<tr>
<td>Trussed Towers</td>
<td>0.85</td>
</tr>
</tbody>
</table>

#### Kzt – Topographic Factor

![Diagram of escarpment and 2-D ridge or 3-D axisymmetric hill]
External Pressure Coefficients

WALLS AND ROOFS (MWFRS)
## Internal Pressure Coefficients

<table>
<thead>
<tr>
<th>Enclosure Classification</th>
<th>$(GC_{pl})$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Open Buildings</td>
<td>0.00</td>
</tr>
<tr>
<td>Partially Enclosed Buildings</td>
<td>+0.55, -0.55</td>
</tr>
<tr>
<td>Enclosed Buildings</td>
<td>+0.18, -0.18</td>
</tr>
</tbody>
</table>

- **Open**
- **Enclosed**
- **Partially Enclosed**
DETERMINING SEISMIC FORCES
Seismic Hazard Map
Seismic Design

Equivalent Lateral Force Procedure per ASCE 7

\[ V = C_s \times W \]

- \( V \) = Seismic Base Shear
- \( C_s \) = Seismic Response Coefficient
- \( W \) = Seismic Mass
Seismic Design Parameters

- Ground Motion
- Site Class
- Fundamental Period of Structure
- Seismic Use Group and Importance Factor
- Seismic Design Category
- Building Configuration
- Response Modification Factor
Soil Profile & Ground Motion

Harder Soils have larger and shorter accelerations

Soft soils have smaller but longer accelerations
IBC Seismic Design Category

6 Site Categories – A, B, C, D, E, & F
5 Design Categories – A, B, C, D & E

Design Response Spectrum

\[ S_a = \frac{S_{D1}}{T} \]

\[ S_a = \frac{S_{D1} \cdot T_L}{T^2} \]
Site Classification

- A: Hard Rock
- B: Rock
- C: Very Dense Soil, Soft Rock
- D: Stiff Soil (Default)
- E: Soft Clay Soil
- F: Soils Requiring Site Analysis (Basically Garbage)
Seismic Design Category

- Design Category A
- Design Category B
- Design Category C
- Design Category D
- Design Category E

### Table 11.6-1: Seismic Design Category Based on Short Period Response Acceleration Parameter

<table>
<thead>
<tr>
<th>Value of $S_{pg}$</th>
<th>Occ. Category 1 or II</th>
<th>Occ. Category III</th>
<th>Occ. Category IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_{pg} &lt; 0.167$</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>$0.167 \leq S_{pg} &lt; 0.33$</td>
<td>B</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>$0.33 \leq S_{pg} &lt; 0.50$</td>
<td>C</td>
<td>C</td>
<td>D</td>
</tr>
<tr>
<td>$0.50 \leq S_{pg}$</td>
<td>D</td>
<td>D</td>
<td>D</td>
</tr>
</tbody>
</table>

### Table 11.6-2: Seismic Design Category Based on 1-S Period Response Acceleration Parameter

<table>
<thead>
<tr>
<th>Value of $S_{pl}$</th>
<th>Occupancy Category 1 or II</th>
<th>Occupancy Category III</th>
<th>Occupancy Category IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_{pl} &lt; 0.067$</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>$0.067 \leq S_{pl} &lt; 0.133$</td>
<td>B</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>$0.133 \leq S_{pl} &lt; 0.20$</td>
<td>C</td>
<td>C</td>
<td>D</td>
</tr>
<tr>
<td>$0.20 \leq S_{pl}$</td>
<td>D</td>
<td>D</td>
<td>D</td>
</tr>
</tbody>
</table>

- NO SPECIAL DETAILING
- MINIMAL SPECIAL DETAILING
- SPECIAL SEISMIC DETAILING
Response Modification R

<table>
<thead>
<tr>
<th>Seismic Force Resisting Systems:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Walls</td>
</tr>
<tr>
<td>Braced Frames</td>
</tr>
<tr>
<td>Moment-Resisting Frame</td>
</tr>
<tr>
<td>Dual Systems</td>
</tr>
<tr>
<td>Cantilevered Column</td>
</tr>
<tr>
<td>Undefined Systems</td>
</tr>
</tbody>
</table>

Dependent on type of structural system and Seismic Design Category

Not all structural systems are allowed in Seismic Design Categories
Seismic Response Coefficient $C_s$

Types of Structural Systems:

- $Cs = \frac{S_{DS}}{(R/I)}$ (Max Value)
- $Cs = \frac{S_{D1}}{T^*(R/I)}$ if $T < T_L$
- $Cs = \frac{S_{D1} T_L}{T^*(R/I)}$ if $T > T_L$
- $Cs_{min} = 0.044 * S_{D1} * I_e$
Seismic Response Coefficient $C_s$

Dependent on type of structural system and Seismic Design Category

Not all structural systems are allowed in Seismic Design Categories

Different Equations Based On Building Period:

\[ C_s = \frac{S_{DS}}{(R/I_e)} \quad (\text{Max Value}) \]

\[ C_s = \frac{S_{D1}}{T(R/I_e)} \quad (T<T_L) \]

\[ C_s = \frac{S_{D1} \cdot T_L}{T^2(R/I_e)} \quad (T>T_L) \]

\[ C_{Smin} = 0.044S_{DS} \cdot I_e > 0.01 \quad (\text{Min Value}) \]
Seismic Response Coefficient $C_s$

Dependent on type of structural system and Seismic Design Category

Not all structural systems are allowed in Seismic Design Categories
Questions?
Lateral
Practice Problems
LATERAL PROBLEMS

For a 2-storey square building in Austin, we want to determine the lateral load requirements that we will have for design.

We need to determine whether wind or seismic controls and figure out the design loads.
LATERAL PROBLEMS

1. Determine the base shears for both wind and seismic load cases.

2. For the controlling load case, determine the base overturning moment.

GIVENS

- 30’x30’ square building
- Uniform wind pressure of 25 psf
- Uniform DL=110 psf at both elevated floors
- Seismic Design Category “A” ($C_s=0.01$)
**SEISMIC BASE SHEAR**

Determine Floor Weight

\[ A_{L2} = A_{Roof} = (30')^2 = 900 \text{ ft}^2 \]

\[ W = 900 \text{ ft}^2 \times 110 \text{ psf} = 99,000 \text{ lb} \]

Determine Seismic Shear

\[ V = C_s W \]

\[ V = 0.01 \times 99,000 \text{ lbs} \]

\[ V_{L1} = V_{Roof} = 0.01 \times 99,000 \text{ lb} \]

\[ = 990 \text{ lb/floor} \]

\[ V_{BASE} = 2 \times 990 \text{ lb} = \text{1980 lb} \]
WIND BASE SHEAR

Determine Tributary Areas

\[ A_{L2} = 30\text{ft} \left( \frac{15\text{ft}}{2} + \frac{10\text{ft}}{2} \right) = 375\text{ft}^2 \]

\[ A_{\text{Roof}} = 30\text{ft} \left( \frac{10\text{ft}}{2} \right) = 150\text{ft}^2 \]

Determine Wind Shears

\[ V = pA_T \]

\[ V_{L2} = 25\text{psf} \times 375\text{ft}^2 = 9375\text{lb} \]

\[ V_{\text{Roof}} = 25\text{psf} \times 150\text{ft}^2 = 3750\text{lb} \]

\[ V_{\text{BASE}} = 9375\text{lb} + 3750\text{lb} = 13125\text{lb} \]

WIND CONTROLS!
OVERTURNING

Calculate Overturning Moment for Wind Load Case

\[ M_O = Vh \]

\[ M_O = (9.4 \text{ kip})(15 \text{ ft}) + (3.8 \text{ kip})(25 \text{ ft}) \]

\[ M_O = 236 \text{ kip} \cdot \text{ft} \]

Calculate Overturning Reactions

\[ R_y = \frac{236 \text{ kip} \cdot \text{ft}}{30 \text{ ft}} = 7.9 \text{ kip} \]
Questions?
Thank You