# Overview of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*

Richard A. Pratt, P.E.

Elmer E. Marx, P.E.

Alaska DOT&PF

# 25 minutes! – Hold on to your seats

ATTACHMENT B - 2007 AGENDA ITEM 7 – T-3

Proposed

AASHTO Guide Specifications for LRFD Seismic Bridge Design

Subcommittee for Seismic Effects on Bridges
T-3

Prepared by:

Roy A. Imbsen
Imbsen Consulting

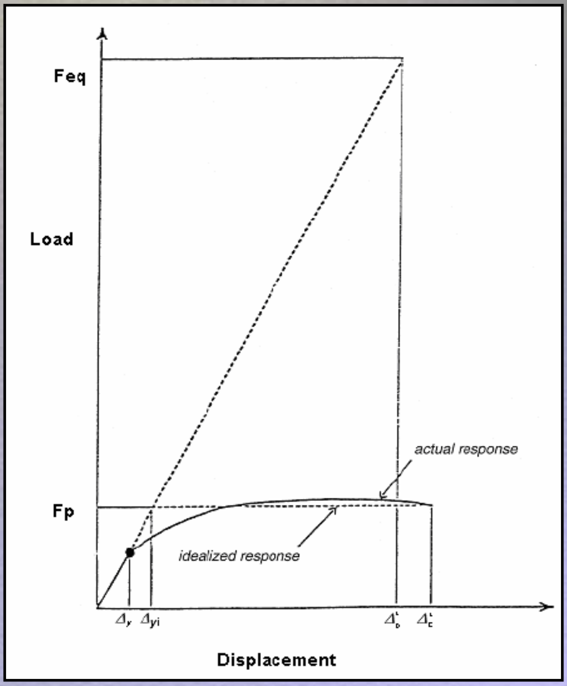
May 2007

# Seismic Design Assumptions

- *Equal Displacement Theory*: The **displacements** calculated for a non-linear structure are suitably close to the **displacements** calculated for the same structure based upon its initial elastic stiffness

# Seismic Design Assumptions

- Current seismic design practice is **force-based** using the elastic seismic forces that have been reduced by a Response Modification Factor (R factor)

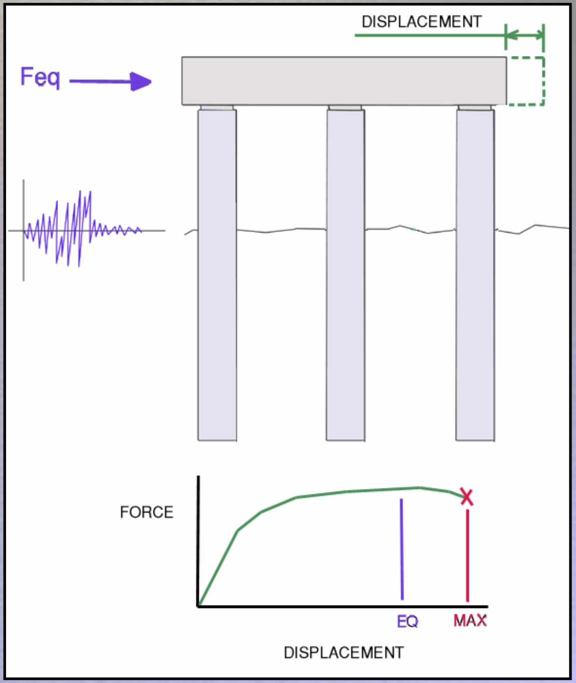



- But it is the **displacements** that are “accurately” predicted not the member forces

# Displacement-Based Approach

- The tools currently exist that allow us to explicitly compare the design seismic displacement demand to the nominal displacement capacity

- Large plastic displacements can be achieved provided that brittle and premature failure modes can be prevented (i.e. *to obviate failure* according to Henry Petroski)



# Some Failure Modes to Obviate

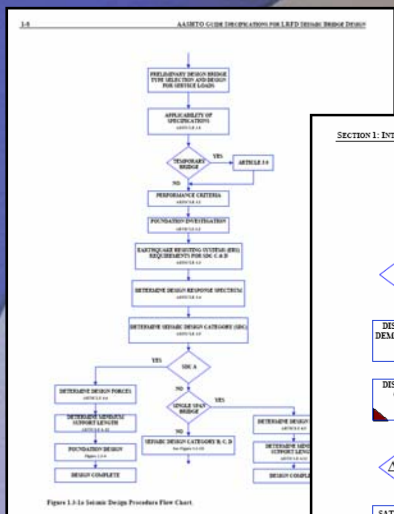
- Unseating of the spans
- Premature tension failure in reinforcing steel
- Column shear failure
- Sudden loss of concrete confinement
- Buckling of longitudinal reinforcing

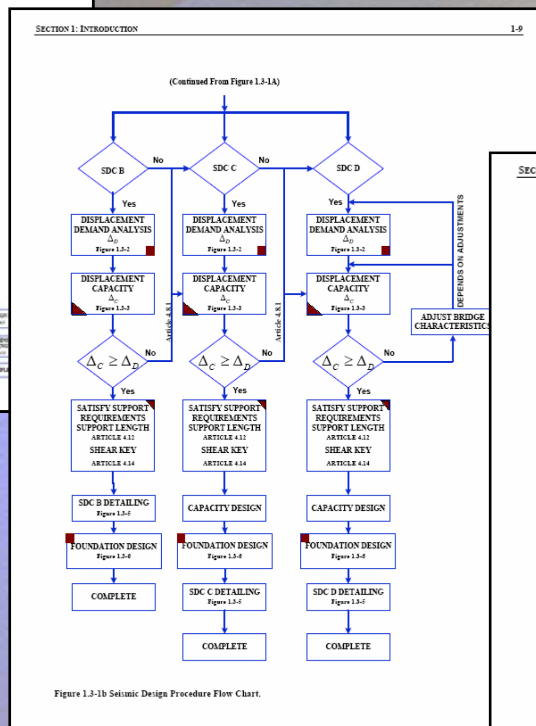
# Where to Start?

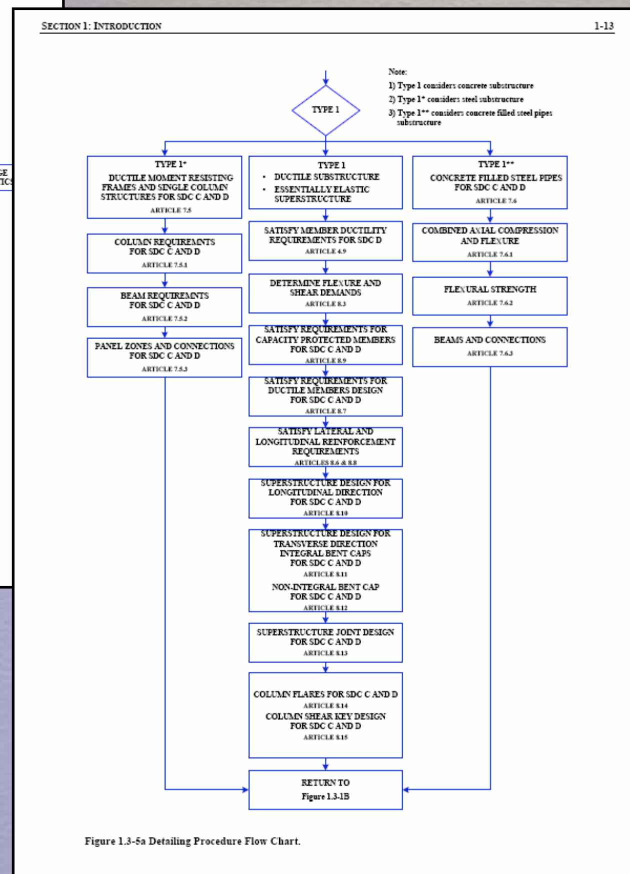
- Design the bridge piers for all applicable non-seismic AASHTO LRFD load combinations

- For all but the most severe seismic areas, the guide specifications are used to verify satisfactory seismic performance assuming appropriate detailing is provided (e.g. seat width, column confinement, column shear capacity, etc.)

# Seismic Design Flow Charts





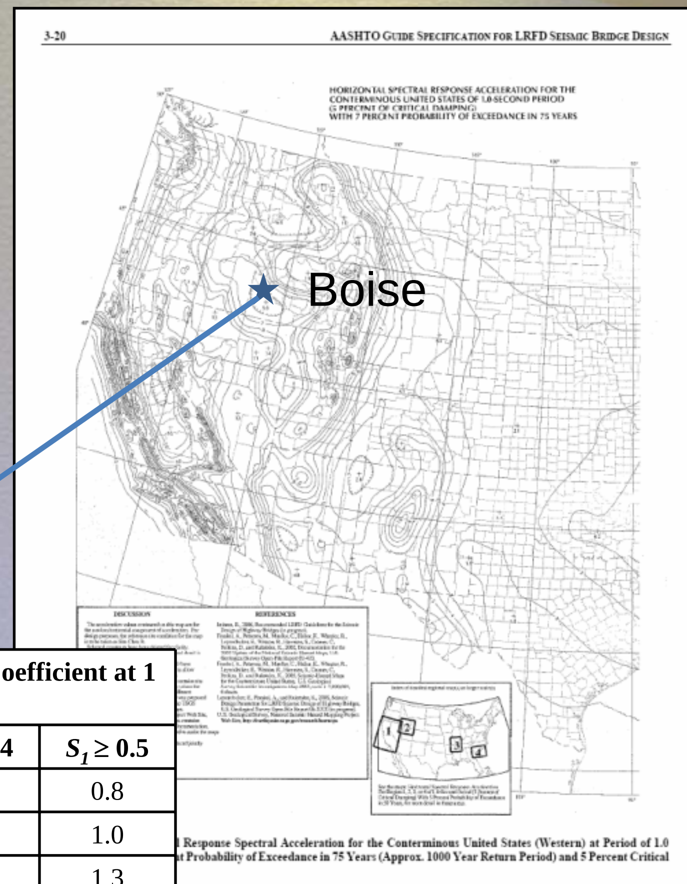



# Seismic Design Categories (SDC)

| **Value of $S_{D1} = F_v S_1$** | **SDC** |
|---|---|
| $S_{D1} < 0.15$ | **A** |
| $0.15 \le S_{D1} < 0.30$ | **B** |
| $0.30 \le S_{D1} < 0.50$ | **C** |
| $0.50 \le S_{D1}$ | **D** |

# 1000-Year Seismic Hazard

- Both the seismic guide specification and the AASHTO LRFD Bridge Design Specifications have adopted the new 1000-year seismic hazard

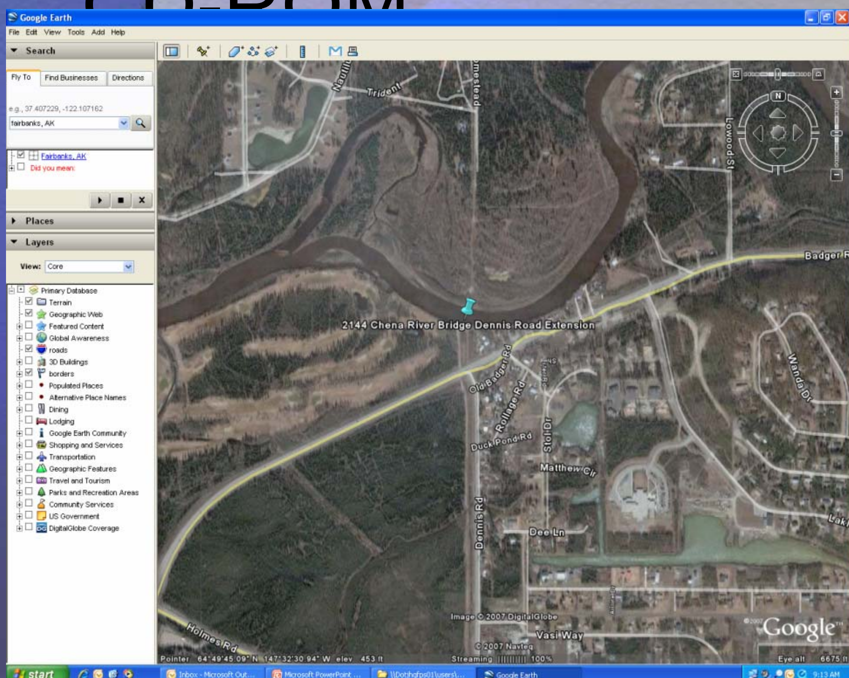


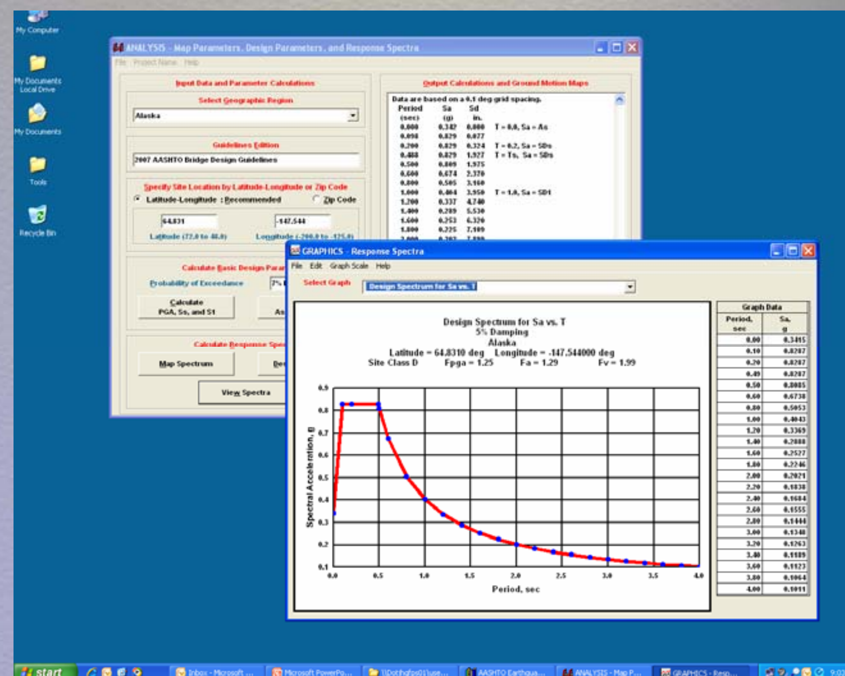

| Site Class | Mapped Spectral Response Acceleration Coefficient at 1 Second Periods | | | | |
|---|---|---|---|---|---|
| | $S_1 \leq 0.1$ | $S_1 = 0.2$ | $S_1 = 0.3$ | $S_1 = 0.4$ | $S_1 \geq 0.5$ |
| **A** | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| **B** | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| **C** | 1.7 | 1.6 | 1.5 | 1.4 | 1.3 |
| **D** | 2.4 | 2.0 | 1.8 | 1.6 | 1.5 |
| **E** | 3.5 | 3.2 | 2.8 | 2.4 | 2.4 |
| **F** | *a* | *a* | *a* | *a* | *a* |

Table notes: Use straight line interpolation for intermediate values of $S_1$, where $S_1$ is the spectral acceleration coefficient at 1.0 sec. obtained from the ground motion maps.

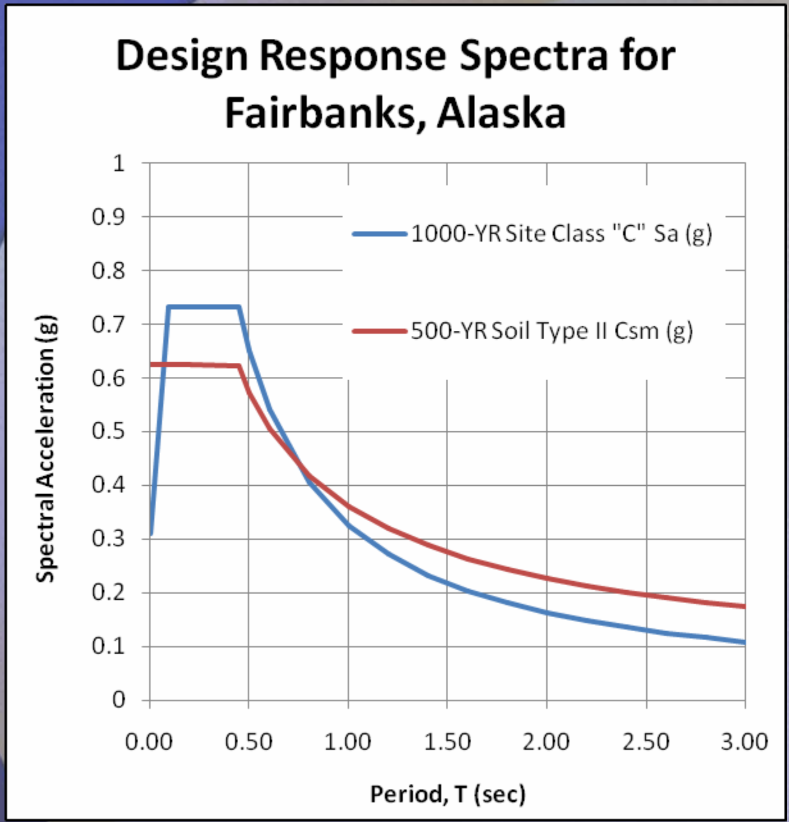
# 1000-Year Seismic Hazard

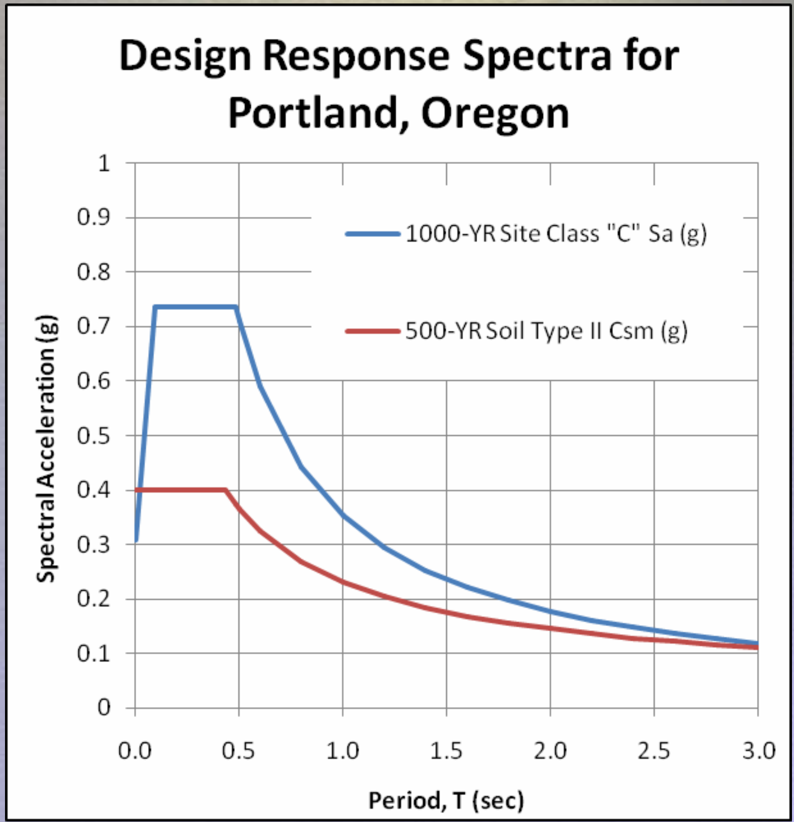
- Google Earth & AASHTO Ground Motion CD-ROM



# Seismic Hazard Comparison

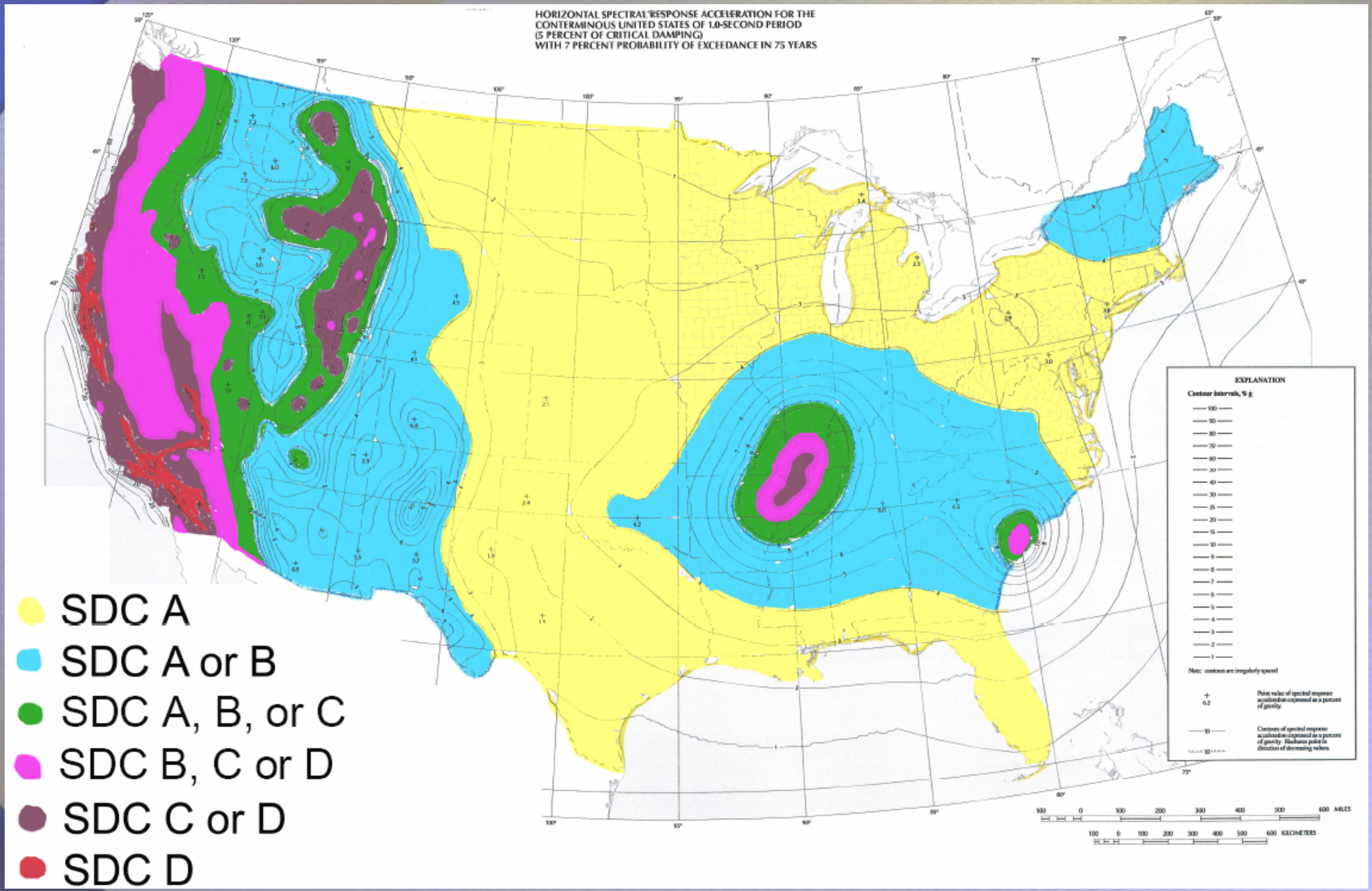


# Seismic Hazard Comparison

- For a T = 1 second period structure with Soil Profile Type II / Site Class C the old 500-year and the new 1000-year spectral demands:
  - Boise $C_{sm}$ ~ 0.05 $S_{D1}$ ~ 0.10 SDC A
  - Butte $C_{sm}$ ~ 0.20 $S_{D1}$ ~ 0.20 SDC B
  - Portland $C_{sm}$ ~ 0.25 $S_{D1}$ ~ 0.35 SDC C
  - Fairbanks $C_{sm}$ ~ 0.35 $S_{D1}$ ~ 0.35 SDC C

# Seismic Design Categories (SDC)



# Seismic Design Categories (SDC A)

- **SDC A** $S_{D1} < 0.15$ $[\mu_D < 1]$
- No explicit seismic analysis required
- Provide minimum bearing seat width
- Provide prescriptive substructure-to-superstructure connections (15% to 25% of the dead load reaction)

- *Also applies to single span beam-slab bridges*

# Seismic Design Categories (SDC B)

- **SDC B** $0.15 < S_{D1} < 0.30$ $[\mu_D < 2]$
- The ERS (TYPE 1) and ERE (column hinges) are assumed
- Modeling is required to determine the seismic displacement demand and it is compared to the *implicit*, *closed-form* displacement capacity
- Column shear design in plastic hinge zone for *minimum* of plastic shear or elastic EQ force demand
- Prescriptive ductile detailing

# Seismic Design Categories (SDC C)

- **SDC C** 0.30 < $S_{D1}$ < 0.50 [$\mu_D$ < 3]
- The ERS (TYPE 1) and ERE (column hinges) are assumed
- Modeling is required to determine the seismic displacement demand and it is compared to the *implicit*, *closed-form* displacement capacity
- Column shear design in plastic hinge zone for plastic shear (capacity protected design)
- Prescriptive and capacity-design detailing

# Seismic Design Categories (SDC D)

- **SDC D** $0.50 < S_{D1}$ [$\mu_D <$ ERE limit]
- The ERS and ERE must be identified
- Modeling is required to determine the seismic displacement demand and it is compared to the *explicit, push-over* displacement capacity
- Column shear design in plastic hinge zone for plastic shear (capacity protected design)
- *Explicit* and capacity-design detailing

# Earthquake Resisting System (ERS)

- The “global” seismic response system
  - **Type 1: ductile substructure elements such as plastic hinges in R/C columns**
  - Type 2: ductile end diaphragms in steel superstructures
  - Type 3: seismic isolation bearings

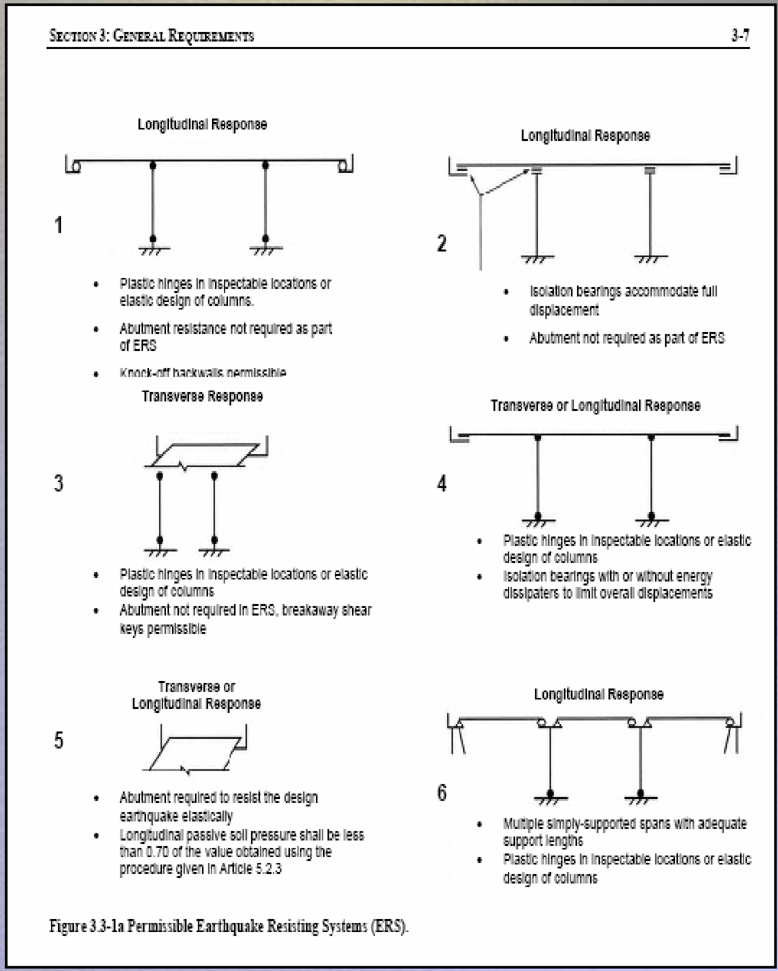
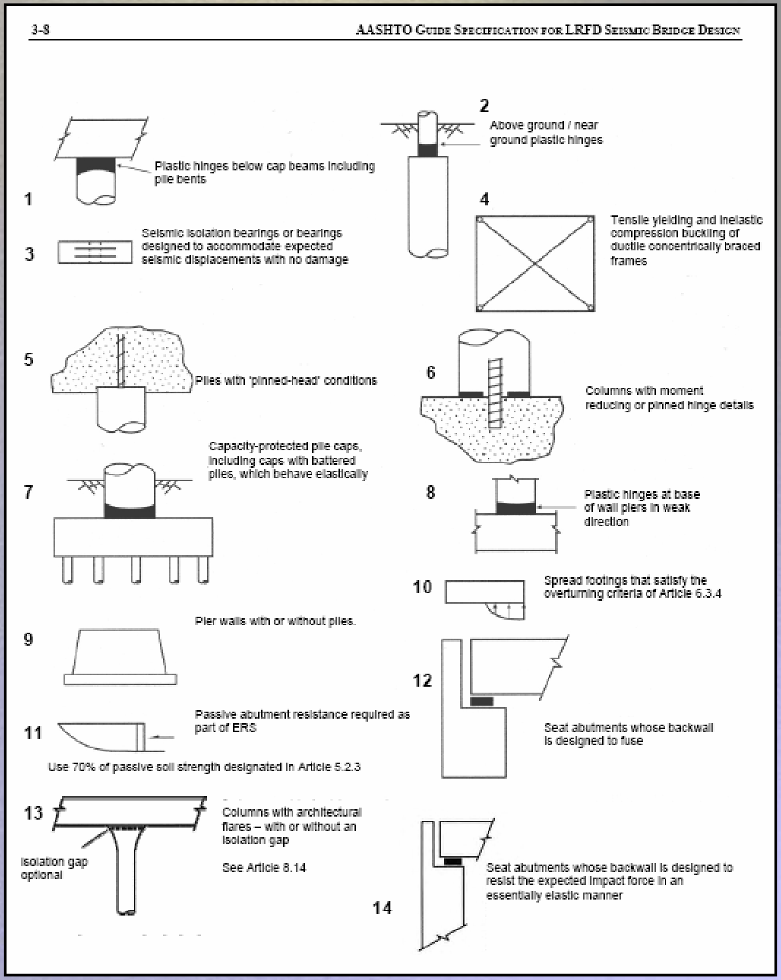


Figure 3.3-1a Permissible Earthquake Resisting Systems (ERS).

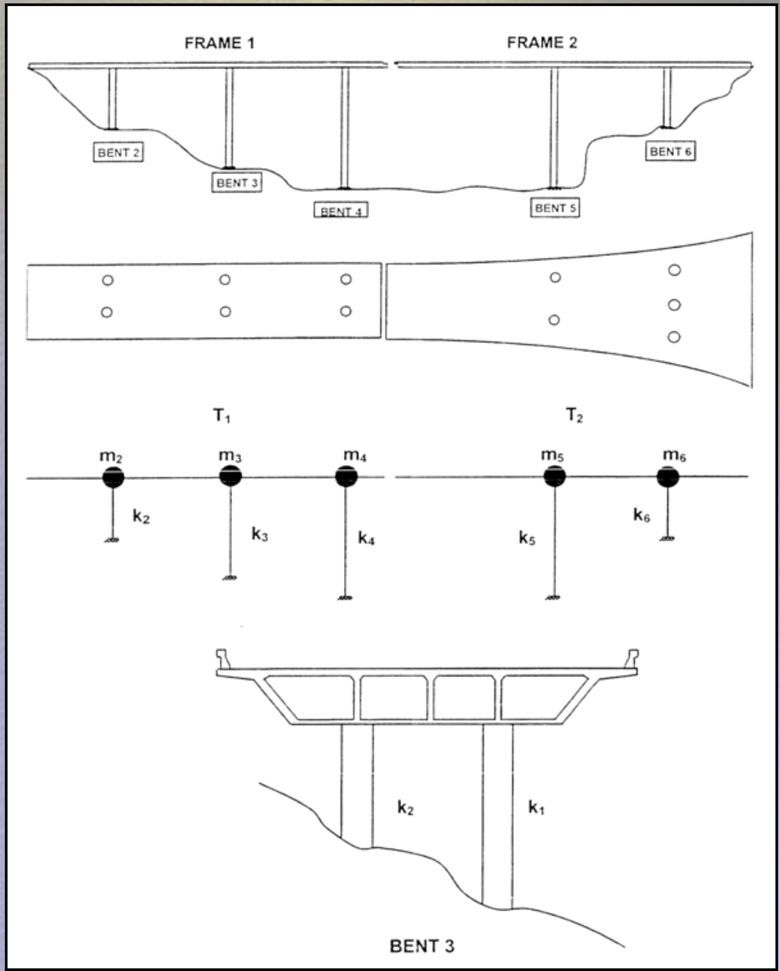
# Earthquake Resisting Element (ERE)

- Various earthquake resisting elements are identified as:
  - **Permissible**
  - **Requires Owner's Approval**
  - Not Recommended



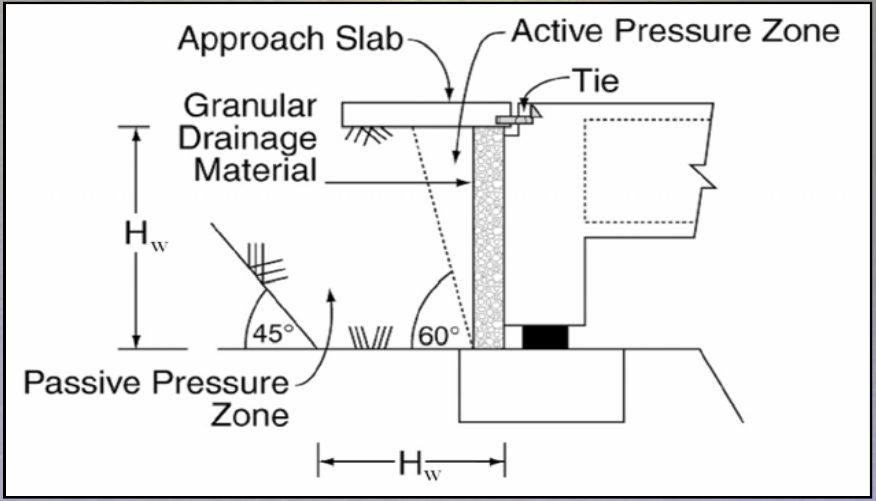
# Modeling Requirements/Guidance

- Relative / balanced stiffness considerations
- Mathematical modeling requirements
- Analysis methodology selection criteria

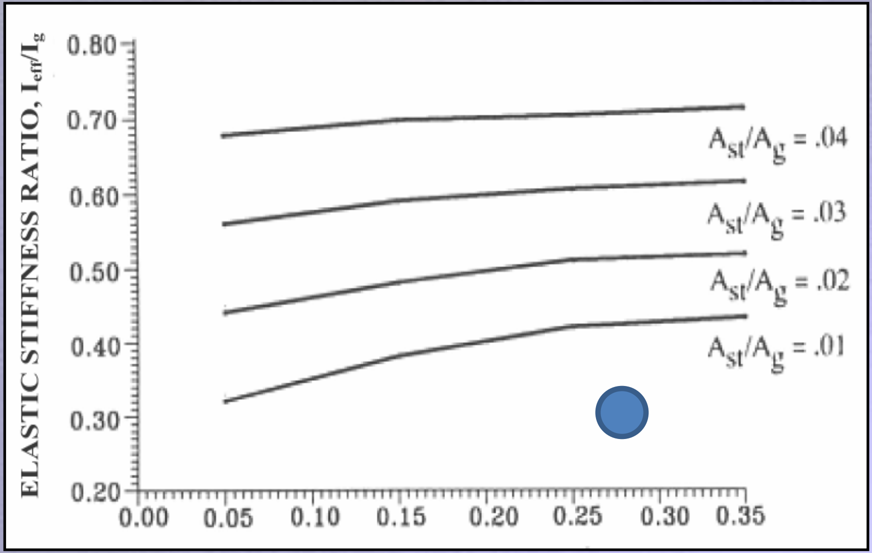


# Modeling Requirements/Guidance

- Methods for incorporating foundation stiffness and boundary conditions




- Effective stiffness of members (SDC B)



# Modeling Requirements/Guidance

- Displacement magnification, $R_d$, for short-period, inelastic structures

$$R_d = \left(1 - \frac{1}{\mu_D}\right)\frac{T^*}{T} + \frac{1}{\mu_D} \geq 1.0 \quad \text{for} \quad \frac{T^*}{T} > 1.0 \qquad (4.3.3\text{-}1)$$

$$R_d = 1.0 \quad \text{for} \quad \frac{T^*}{T} \leq 1.0 \qquad (4.3.3\text{-}2)$$

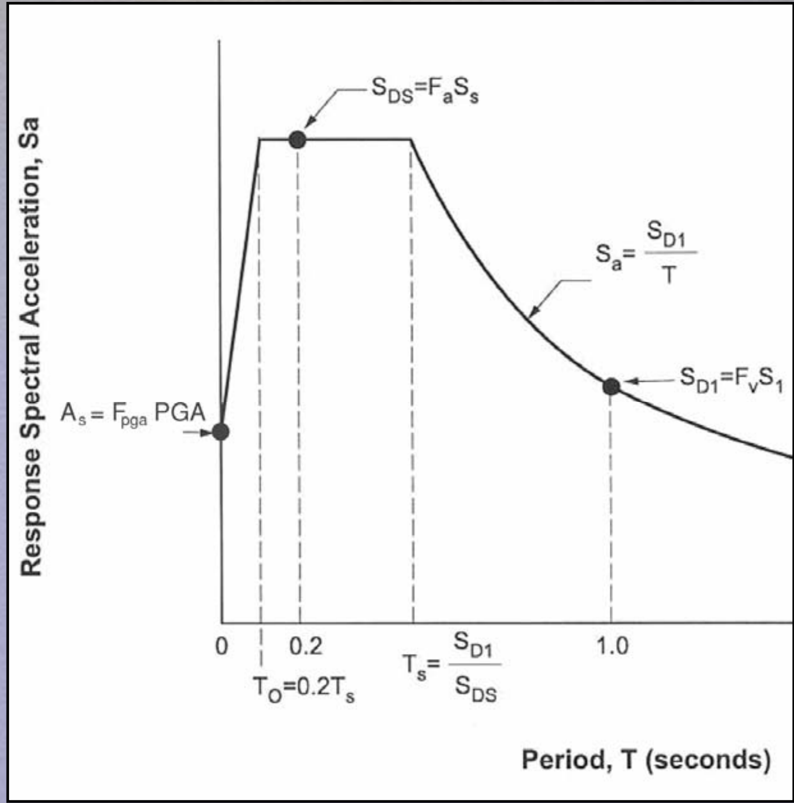
in which:

$$T^* = 1.25T_s \qquad (4.3.3\text{-}3)$$

where:

$\mu_D$ = maximum local member displacement ductility demand
= 2 for SDC B
= 3 for SDC C
= determined in accordance with Article 4.9 for SCD D. In lieu of a detailed analysis, $\mu_D$ may be taken as 6.

$T_s$ = period determined from Article 3.4.1 (sec.)



# Displacement-Based Design

- For SDC B and C, *closed-form* member displacement capacity equations are available

SDC B: $\Delta^{L}_{C}=0.12H_o(-1.27\mathbf{ln}(x)-0.32)>0.12H_o$

SDC C: $\Delta^{L}_{C}=0.12H_o(-2.32\mathbf{ln}(x)-1.22)>0.12H_o$

where: $x = \Lambda B_o/H_o$

# Displacement-Based Design

- For a rough check of conventional circular reinforced concrete column sections:

$$\Delta_y \sim 1/3{*}\phi_y(12H_o+9d_b)^2 \sim H_o^2/50B_o$$

where:

$\Delta_y$ = idealized yield displacement (IN)

$H_o$ = contraflexure to plastic hinge distance (FT)

$d_b$ = diameter of longitudinal column bar (IN)

$\phi_y \sim 2.25{*}\varepsilon_y/12B_o$ (1/IN)

$B_o$ = column diameter (FT)

$\varepsilon_y$ = expected yield strain (IN/IN)

$\varepsilon_{su}^{R}$

# Material Properties for Push-Over

- For SDC D, a push-over analysis is required

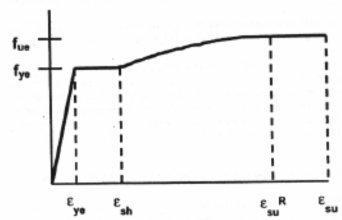
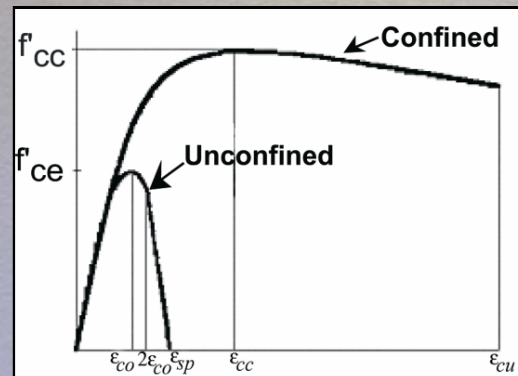
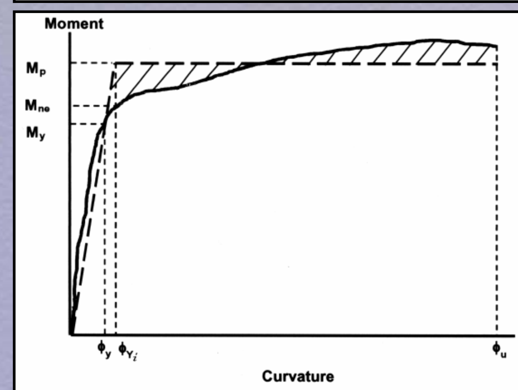



Figure 8.4.2-1 Reinforcing Steel Stress-Strain Model.

Table 8.4.2-1 Stress Properties of Reinforcing Steel Bars.

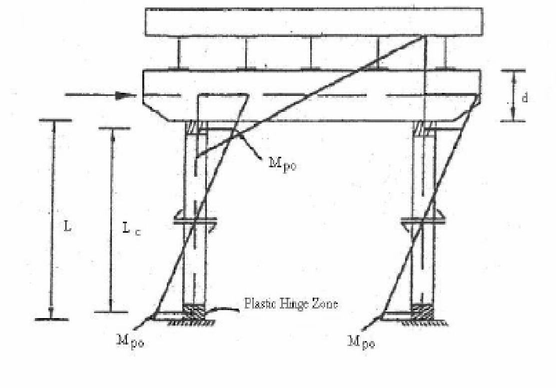
| Property | Notation | Bar Size | ASTM A706 | ASTM A615 Grade 60 |
|---|---|---|---|---|
| Specified minimum yield stress (ksi) | $f_y$ | #3 - #18 | 60 | 60 |
| Expected yield stress (ksi) | $f_{ye}$ | #3 - #18 | 68 | 68 |
| Expected tensile strength (ksi) | $f_{ue}$ | #3 - #18 | 95 | 95 |
| Expected yield strain | $\varepsilon_{ye}$ | #3 - #18 | 0.0023 | 0.0023 |
| Onset of strain hardening | $\varepsilon_{sh}$ | #3 - #8 | 0.0150 | 0.0150 |
| | | #9 | 0.0125 | 0.0125 |
| | | #10 - #11 | 0.0115 | 0.0115 |
| | | #14 | 0.0075 | 0.0075 |
| | | #18 | 0.0050 | 0.0050 |
| Reduced ultimate tensile strain | $\varepsilon_{su}^{R}$ | #4 - #10 | 0.090 | 0.060 |
| | | #11 - #18 | 0.060 | 0.040 |
| Ultimate tensile strain | $\varepsilon_{su}$ | #4 - #10 | 0.120 | 0.090 |
| | | #11 - #18 | 0.090 | 0.060 |



# Capacity Protection Philosophy

- Generally, the overstrength plastic moment demand ($M_{po} = \lambda_{mo} M_p$) and the associated forces acting on a member are the greatest forces that the member can experience




(b) Transverse Response for Dual Column Pier.

Figure 4.11.2-1 Capacity Design of Bridges Using Overstrength Concepts.

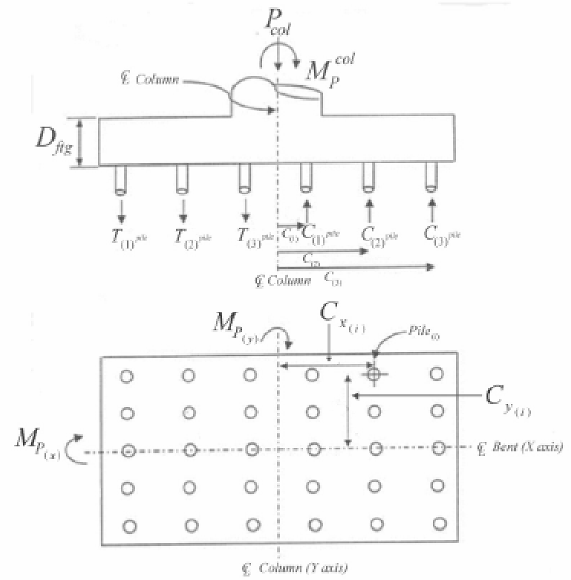
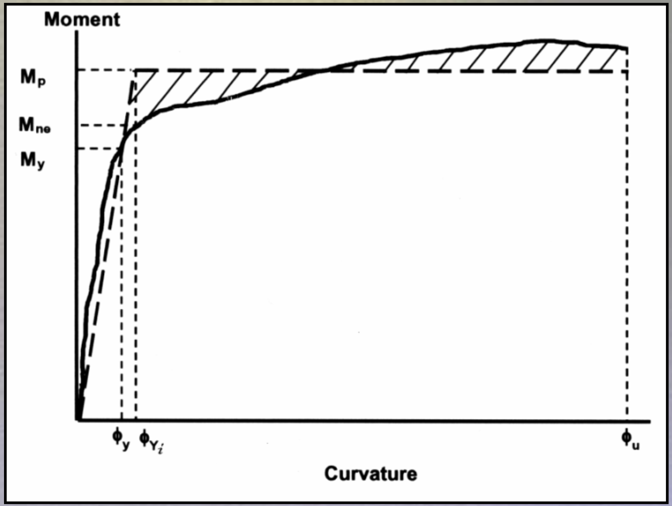



Figure 6.4.2-1 Simplified Model for Pile Foundations in Competent Soil.

# Capacity Protection Philosophy

- $\lambda_{mo}$ – Overstrength factors are material dependant and are applied in addition to the expected material properties
- $\phi$ – Resistance factors are typically taken as 1 except when noted (e.g. shear)
- $\gamma$ – Load factors for the Extreme Event I load case are all taken as 1




$$M_{po} = \lambda_{mo} M_p \qquad (8.5\text{-}1)$$

where:

$M_p$ = idealized plastic moment capacity of reinforced concrete member based upon expected material properties (kip-ft.)

$M_{po}$ = overstrength plastic moment capacity (kip-ft.)

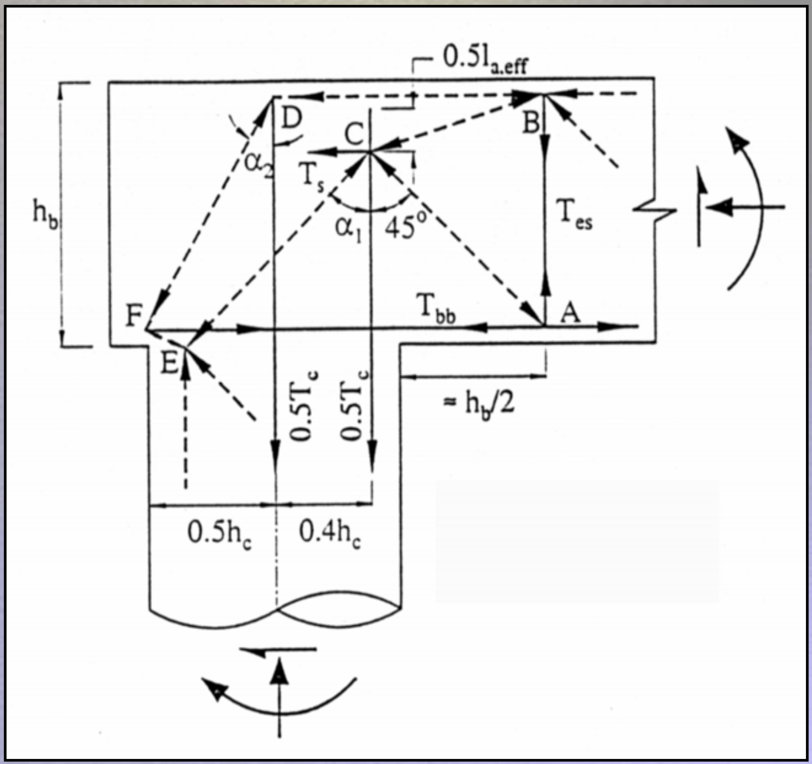
$\lambda_{mo}$ = overstrength magnifier
= 1.2 for ASTM A 706 reinforcement
= 1.4 for ASTM A 615 Grade 60 reinforcement

# Material Level Design and Detailing

- New provisions for the shear capacity of reinforced concrete members in the plastic hinge region are provided
- The provisions are based upon Caltrans practice and have been found to be conservative for most conditions
- Include factors to account for ductility demand and axial loads

# Material Level Design and Detailing

- New column-cap beam joint design provisions have been provided
  - Design requirements are somewhat prescriptive
  - Equations are based upon experimentally verified strut-and-tie models
  - Ensure that hinging occurs in the designated hinge region and not in the joint



# Summary

- Displacement-based design approach (except for ductile steel members)
- 1000-year seismic hazard
- Closed-form displacement equations
- “No-Analysis” for SDC A
- Displacement check and detailing for SDC B
- Displacement check and capacity design for SDC C and D

# Thank You & Questions

Roy Imbsen

NCHRP Manager, Panel and Reviewers

Trial Design States

AASHTO T-3 Members

Technical Review Team

Many Others