# Displacement-Based Seismic Design of Structures

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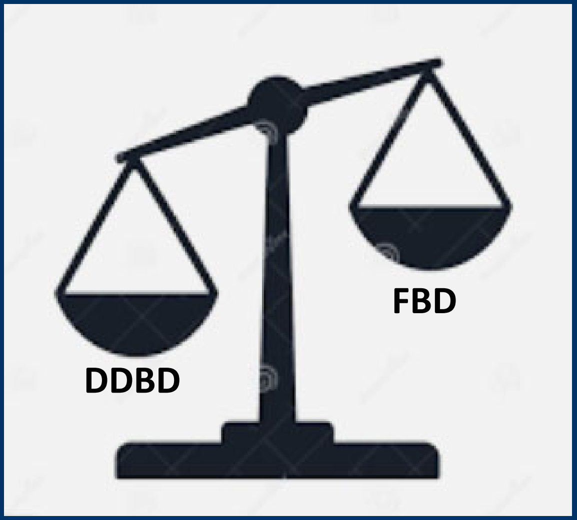
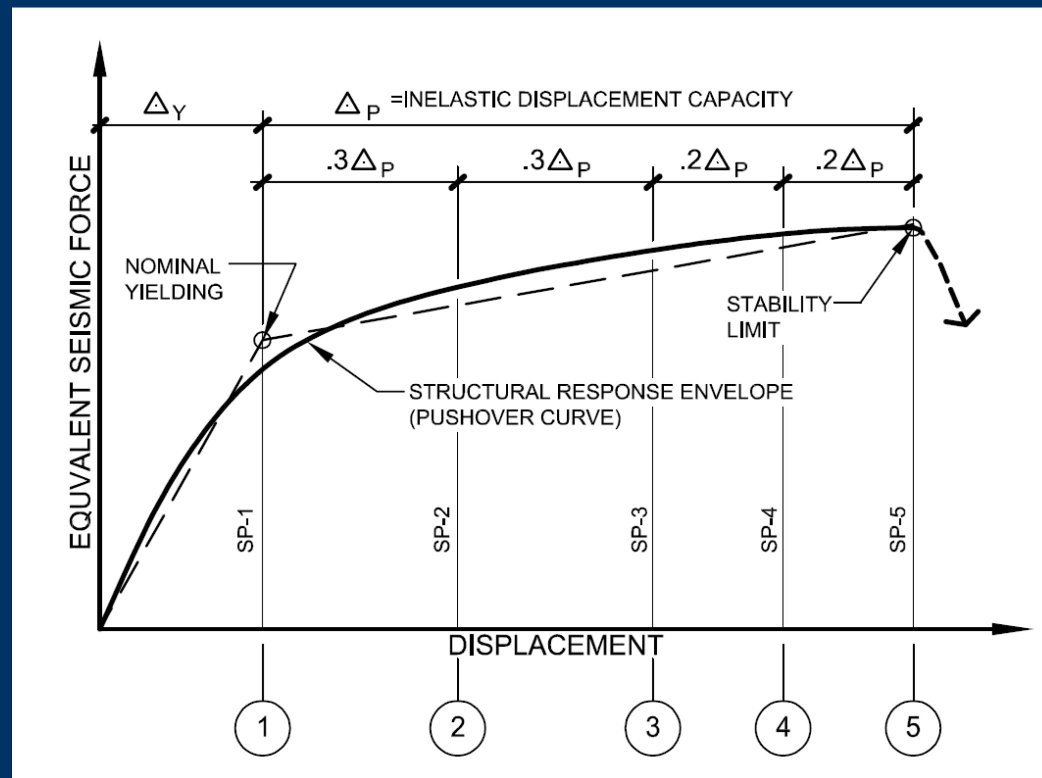
# Outline

- The "Prosecution" of Force-Based Design and the "Case" for Direct Displacement-Based Design (DDBD).
- The Fundamentals of DDBD.
- DDBD for SDOF Systems
- DDBD for more complex systems (Bridges; Structural Walls; Frames)



# FUNDAMENTAL PROBLEM WITH FORCE-BASED DESIGN

- 1. Structural damage (performance level) is related to strain
- 2. Non-structural damage is generally related to drift
- 3. Strain and drift can be integrated to give displacement
- 4. Performance levels can thus be related to displacement
- 5. <u>There is no simple relationship between performance level and strength</u>




**SEAOC Bluebook – 2004 (First 'codification' of DDBD within PBSE Framework)**

# DESIGN TO ELASTIC ACCELERATION SPECTRA (FORCE-BASED DESIGN)

## ASSUMPTIONS (FALLACIES) ARE:

- Elastic force levels (dependent on initial stiffness) are of prime importance
- Elastic stiffness is known at the start of design
- Elastic forces can be reduced by ductility factors, dependent only on material and structural type
- Maximum transient response is the issue. Residual displacement is irrelevant
- Displacements are well estimated by elastic response values

# SPECIFIC PROBLEMS WITH FORCE-BASED DESIGN

- Interdependency of Strength and Stiffness
- Period Calculation
- Ductility Capacity and Force-Reduction Factors
- Ductility of Structural Systems
- Bridges with Unequal Column Heights (or wall buildings with unequal wall lengths)
- Structures with Dual Load Paths (i.e., bridge superstructure and columns)
- Relationship between Elastic and Inelastic Displacement Demand (Displacement Equivalence Rules)

# INTERDEPENDENCY OF STRENGTH AND STIFFNESS

- Member sizes are assumed

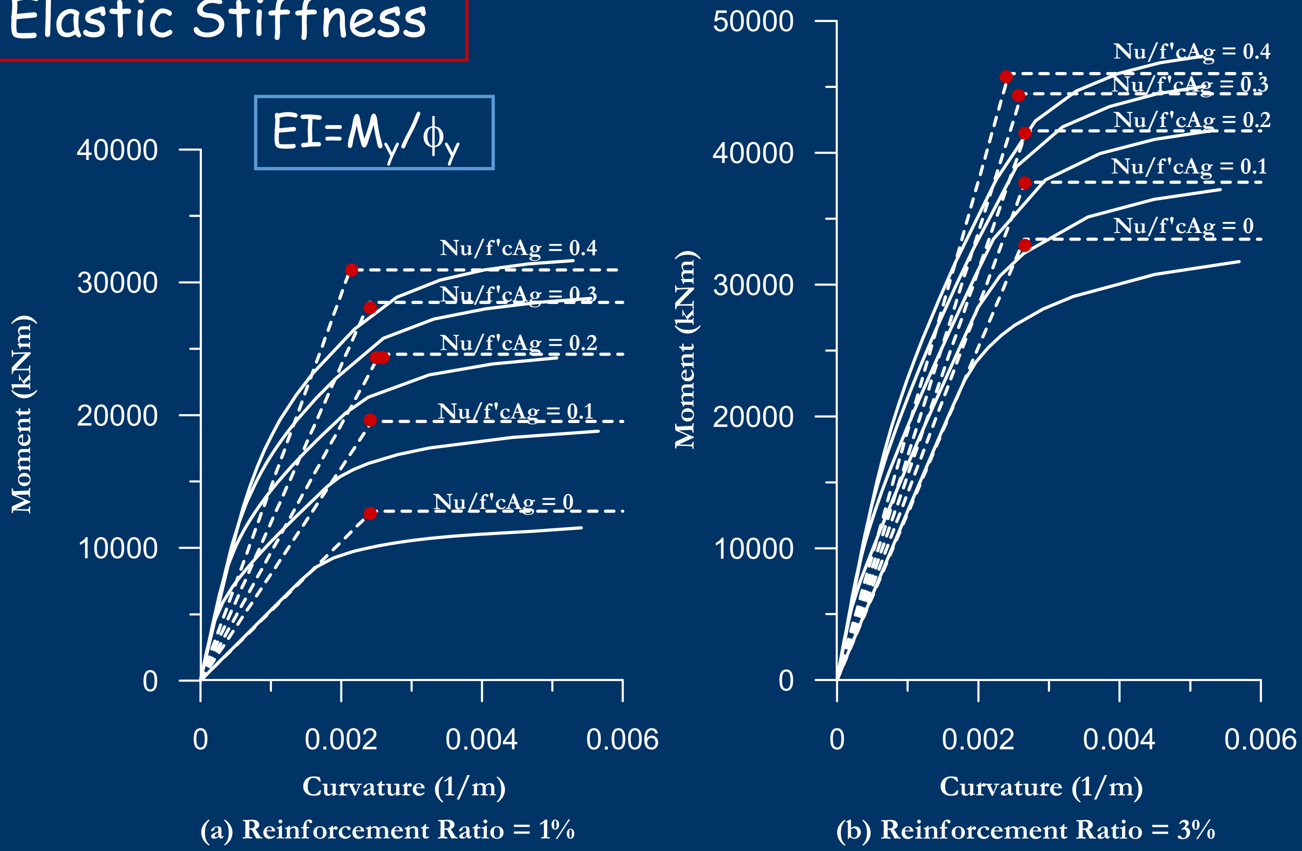
- Member stiffnesses are assumed. In some codes, gross (uncracked) stiffness is assumed.  In others, 50%, or 35 – 70% assumed.

- Periods are based on these assumed stiffnesses. (Note: design forces are reduced by 40% for 0.35Igross vs. 1.0Igross)

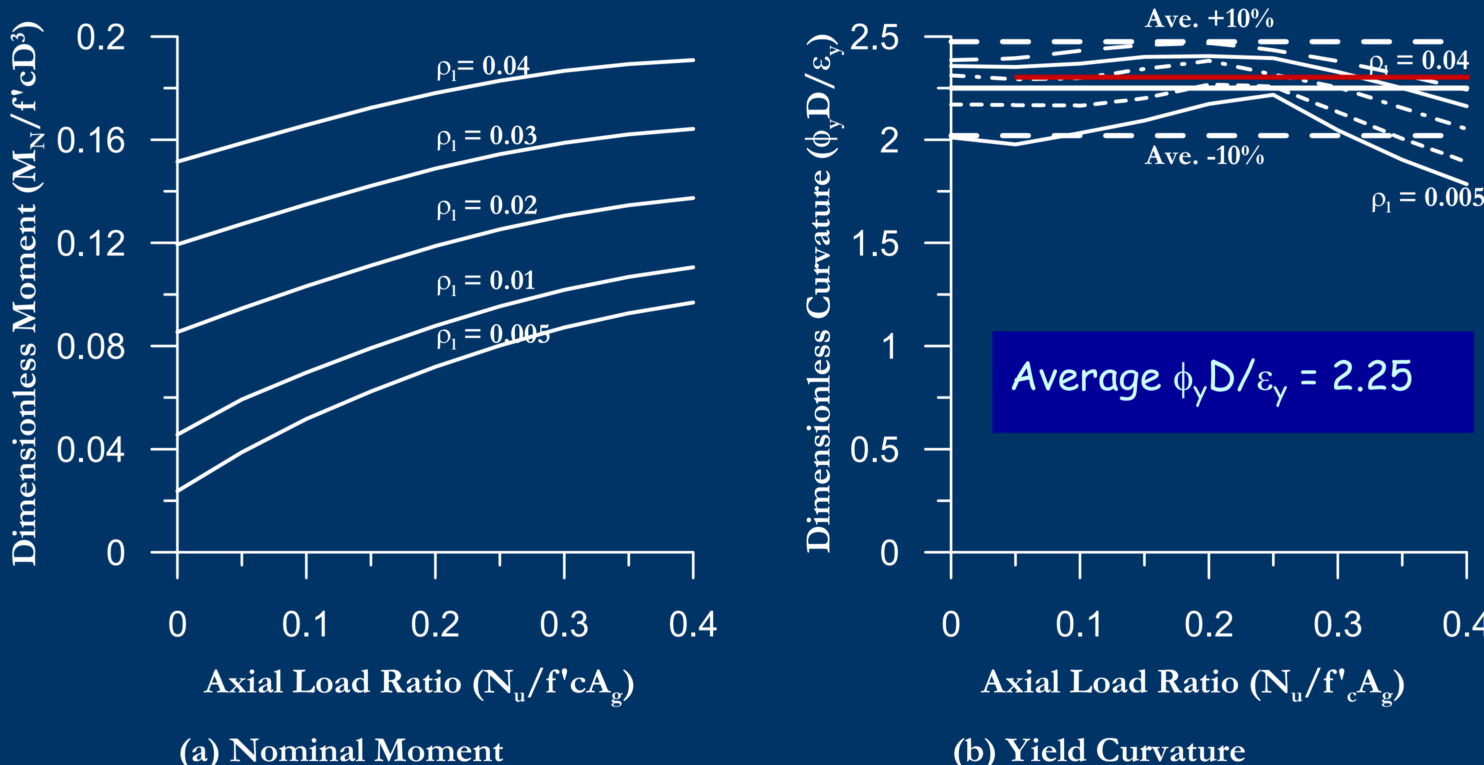
- Forces are distributed to members in proportion to the assumed stiffness

# Elastic Stiffness

$EI = M_y/\phi_y$

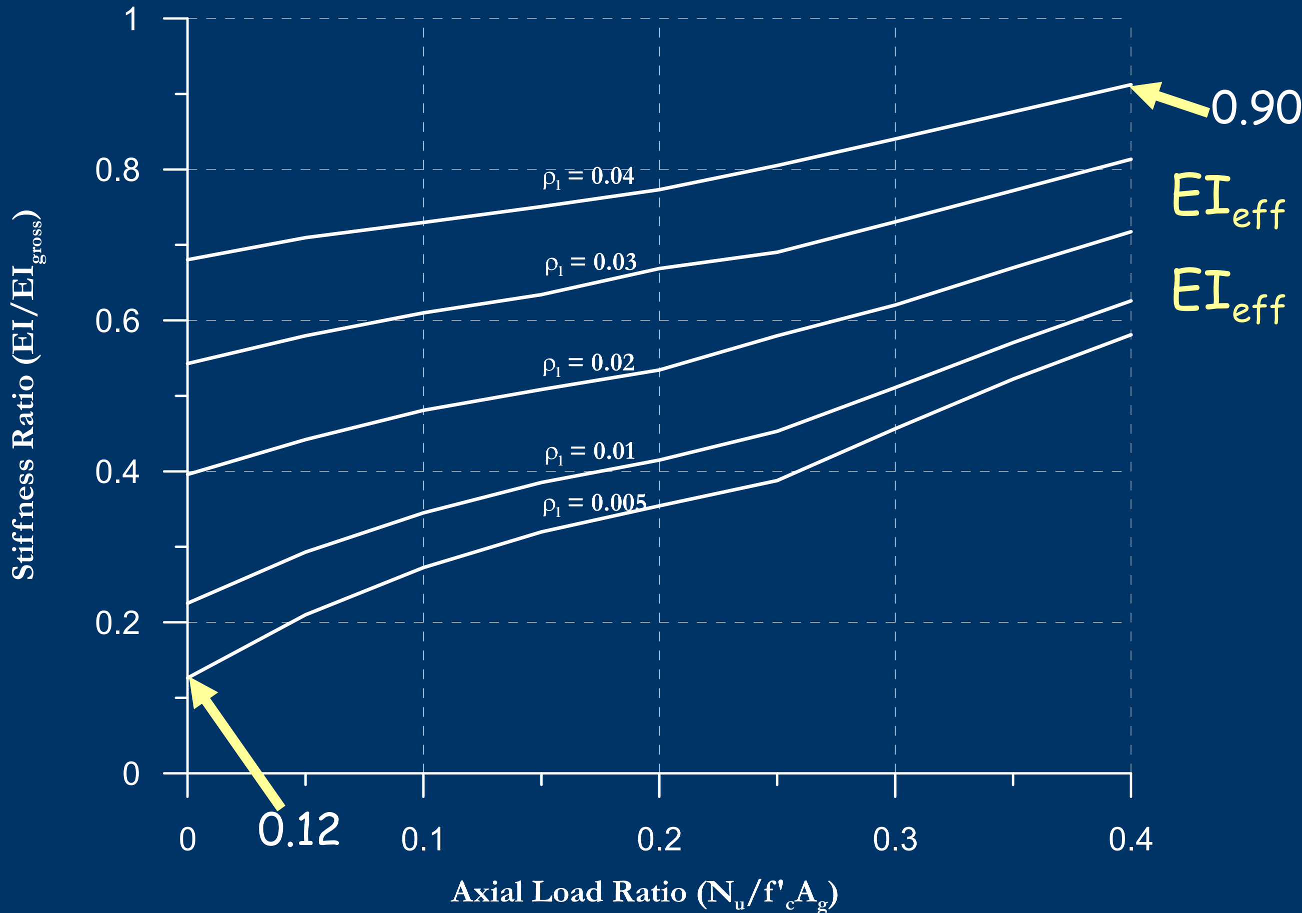



SELECTED MOMENT-CURVATURE CURVES FOR CIRCULAR COLUMNS ($D$=2m, $f'_c$= 35MPa, $f_y$ = 450MPa)




DIMENSIONLESS NOMINAL MOMENT AND YIELD CURVATURE FOR CIRCULAR COLUMNS

STRENGTH VARIES; YIELD CURVATURE IS CONSTANT




$EI_{eff} = M_N/\phi_y$

$EI_{eff}\ /Ei_{gross}=M_N/\phi_y EI_{gross}$

EFFECTIVE STIFFNESS RATIO FOR CIRCULAR COLUMNS

# DIMENSIONLESS YIELD CURVATURES AND DRIFTS

**CURVATURE**

Circular column: $\phi_y = 2.25\varepsilon_y / D$

Rectangular column: $\phi_y = 2.10\varepsilon_y / h_c$

Rectangular cantilever walls: $\phi_y = 2.00\varepsilon_y / l_w$

T-Section Beams: $\phi_y = 1.70\varepsilon_y / h_b$

**DRIFT**

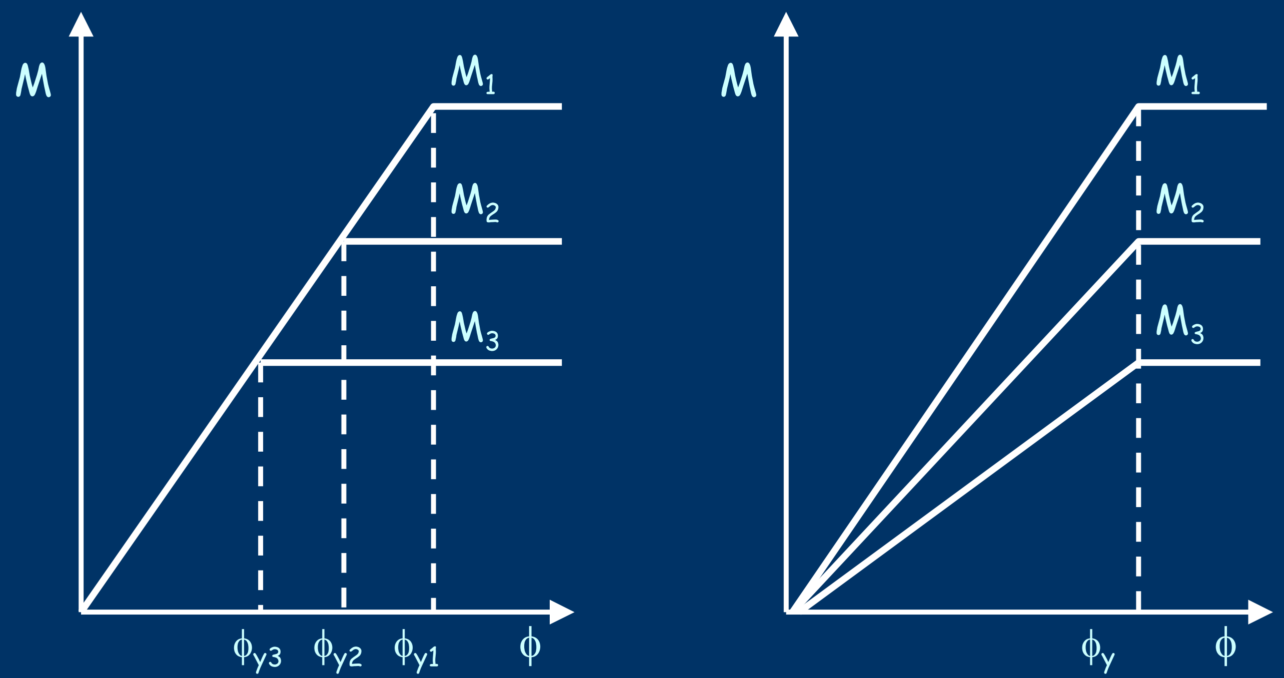
**Concrete Frames:** $\theta_y = 0.5\varepsilon_y \dfrac{l_b}{h_b}$

**Steel Frames:** $\theta_y = 0.65\varepsilon_y \dfrac{l_b}{h_b}$

For SDOF structure, the **Yield Displacement**: $\boldsymbol{\Delta_y = \dfrac{\phi_y H^2}{3}}$ ***or*** $\boldsymbol{\Delta_y = \theta_y H_e}$

# INTERDEPENDENCY OF STRENGTH AND STIFFNESS

Stiffness EI = M/$\phi$




(a) Design assumption (constant stiffness)

(b) Realistic assumption (constant yield curvature)

INFLUENCE OF STRENGTH ON MOMENT-CURVATURE RESPONSE

# FORCE-BASED DESIGN

## ASSUMPTIONS OF SYSTEM DUCTILITY

In current force-based design it is assumed that structural systems have a unique ductility capacity, and hence a unique force-reduction factor

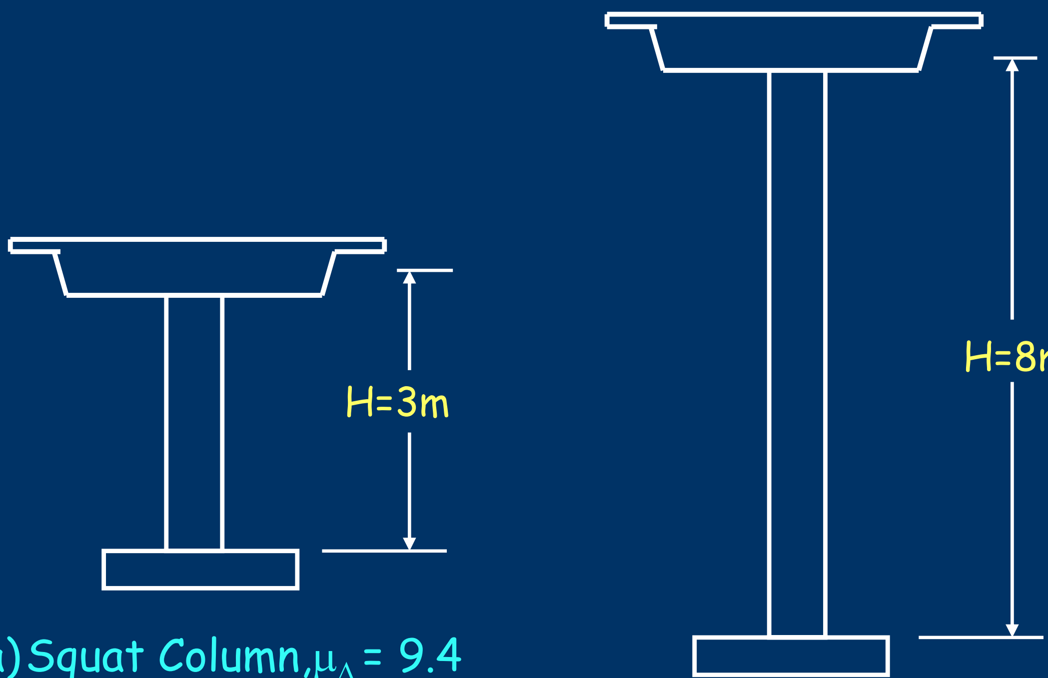
e.g.

Concrete Frame Building: $R_\mu = 6$ (depends on country)

Concrete Wall Building: $R_\mu = 4$ (::)

Concrete Bridge: $R_\mu = 3$ (::)

# CONSIDER BRIDGE COLUMNS OF DIFFERENT HEIGHTS




(a) Squat Column, $\mu_\Delta = 9.4$

(b) Slender Column, $\mu_\Delta = 5.1$

$$\Delta_y = \phi_y H^2 / 3$$

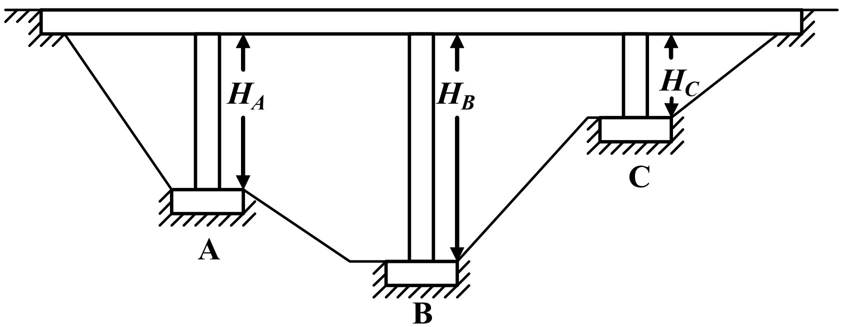
$$\Delta_p = \phi_p L_p H$$

$$\mu_\Delta = \frac{\Delta_y + \Delta_p}{\Delta_y} = 1 + 3\frac{\phi_p L_p}{\phi_y H}$$

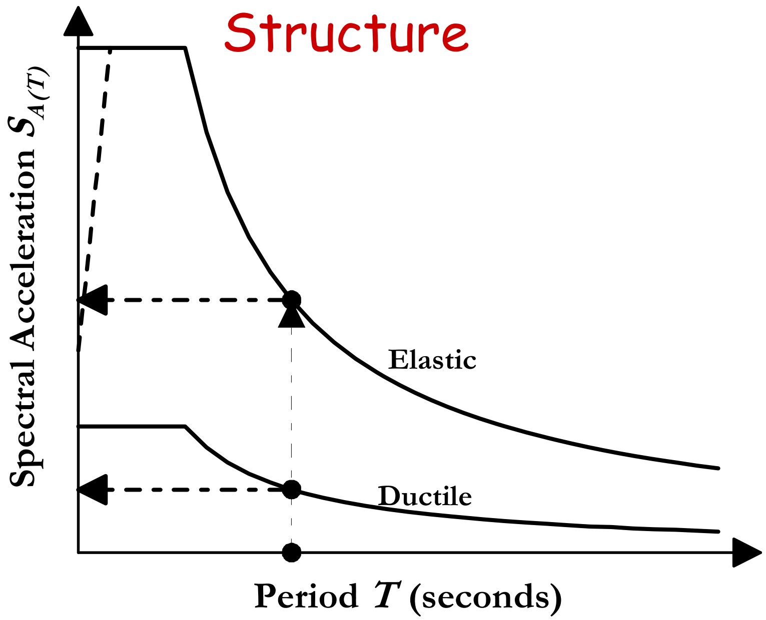
$$L_P = 0.08H + 0.022 d_b f_y$$

Constants: P/f'cAg =0.10, Rebar = 2% long., 0.6% transverse

# Concrete Bridge under Longitudinal Seismic Excitation



Structure



Acceleration Response Spectrum

Stiffness:

$$K_{long} = K_A + K_B + K_C = \sum_{A,B,C} \frac{12EI}{H^3}$$

Period:

$$T = 2\pi\sqrt{m / K_{long}}$$

Base shear force:

$$F = \frac{m \cdot g \cdot S_{A(T)}}{R}$$

Pier Shear Force:

$$F_i = F \frac{K_i}{K_{long}}$$

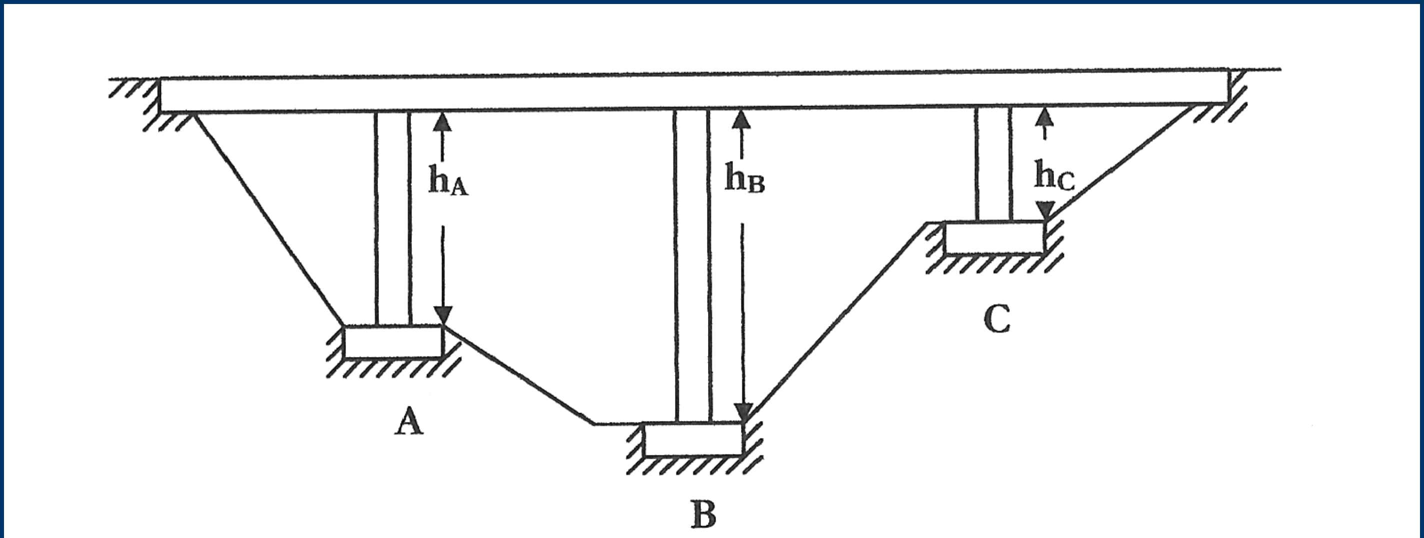
Design Displacement:

$$\Delta_{long} = \frac{T^2}{4\pi^2} \cdot S_{A(T)} \cdot g$$



What value of EI? What value of ductility?

# STRUCTURES WITH UNEQUAL COLUMN HEIGHTS
# BRIDGE CROSSING A VALLEY




Force-Based Design: Column Stiffness $$K_i = C_1 EI_{i.eff} / h_i^3$$

Force-Based Design: Column Moments: $$M_{Bi} = C_2 V_i h_i = C_1 C_2 EI_{i.eff} / h_i^2$$

Short columns need highest long.rebar: stiffness increases: higher moment!

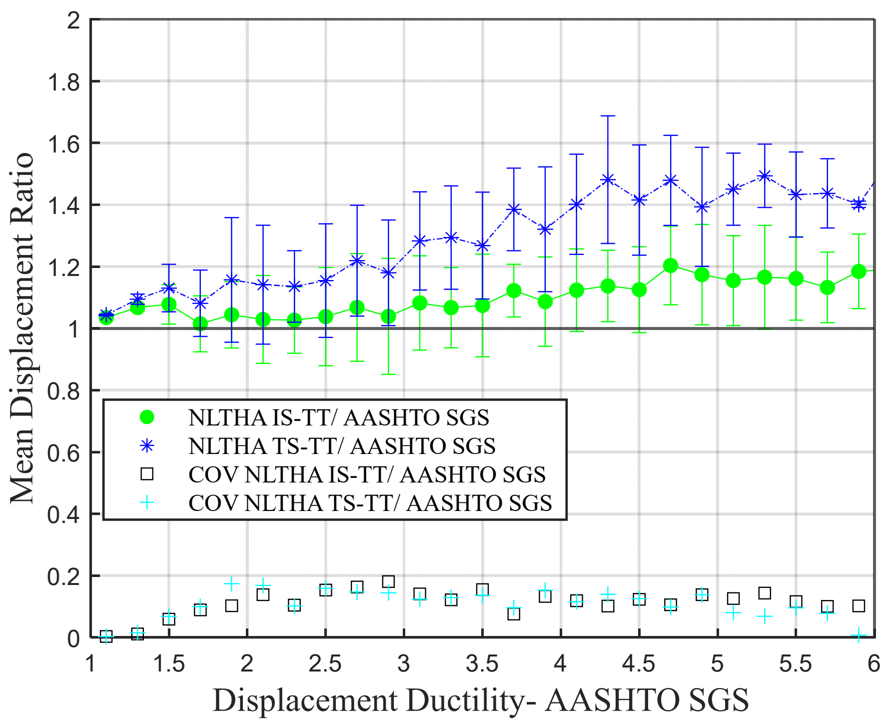
# DIRECT DISPLACEMENT-BASED DESIGN

How displacements are considered in Force-Based Design: Displacements are checked (<u>or should be! Often they are not</u>) at the end of the design process, using approximate stiffnesses and rules, to ensure displacement limits are not exceeded. Note that there is NO direct relationship between force (strength) and damage.

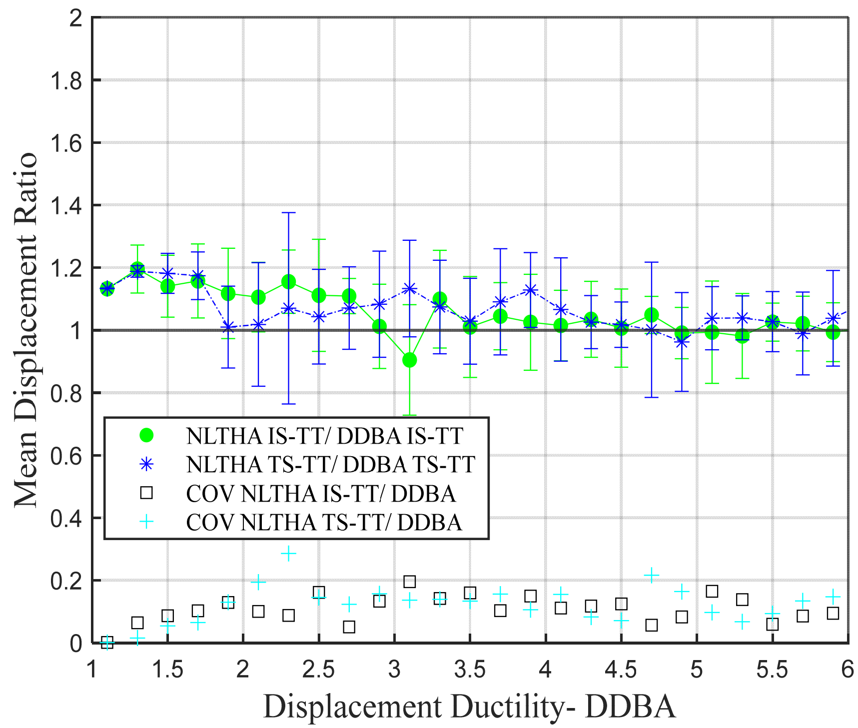
Direct Displacement-Based Design (DDBD): Structure is designed so that it *ACHIEVES* (rather than is bounded by) a given performance limit state, represented by displacement limits.

# Displacement Equivalence Rules (from Martinez)

1. Mean Response NLTHA vs SGS AASHTO




2. Mean Response NLTHA vs DDBA

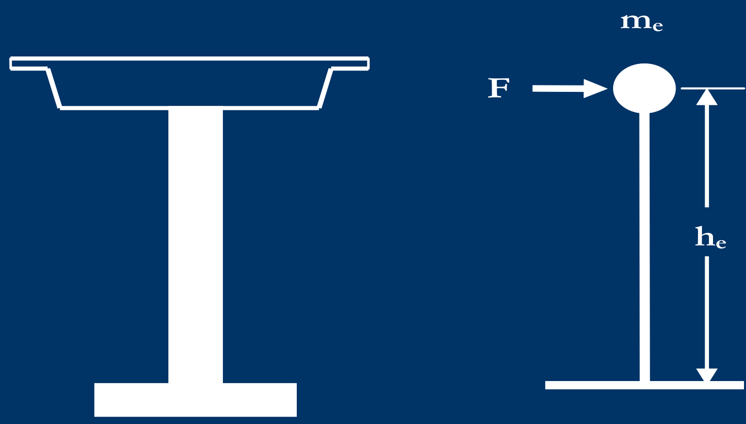


# FUNDAMENTAL DIFFERENCE BETWEEN FORCE BASED AND DISPLACEMENT BASED DESIGN

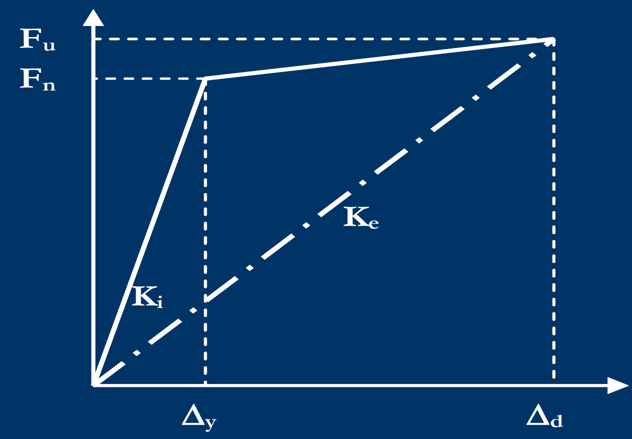
Force-Based uses an estimate of initial stiffness and elastic damping (typically 5%) to estimate required strength, and final inelastic displacement. The strength is distributed to members in proportion to elastic stiffness

Direct Displacement-Based Design uses a calculated value for secant stiffness, equivalent viscous damping, and a damage-related limit-state displacement to determine required strength. This strength is NOT distributed to members in proportion to elastic stiffness.

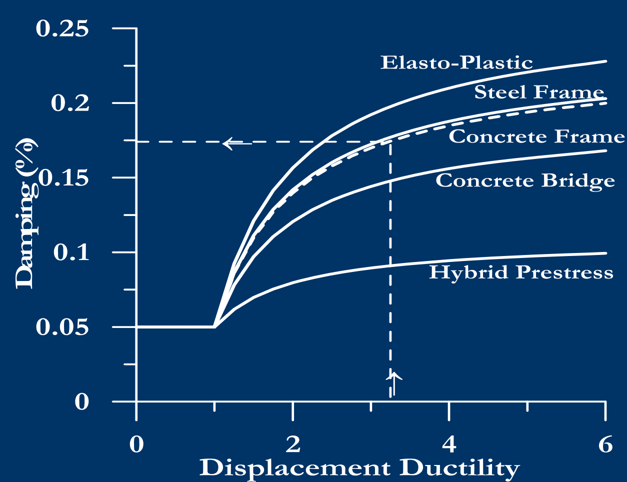
# FUNDAMENTALS OF DDBD



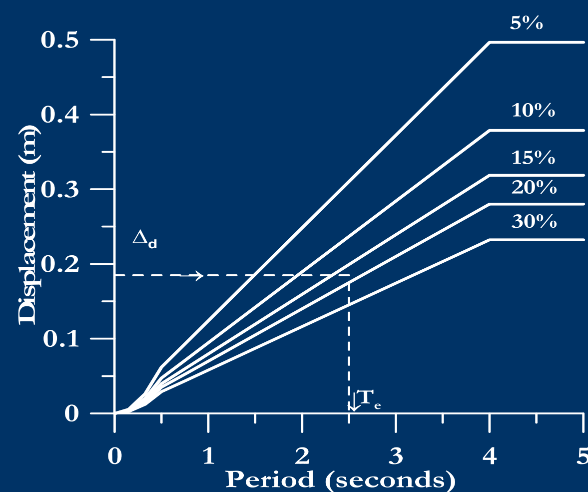

(a) SDOF Simulation




(b) Effective Stiffness $K_e$




(c) Equivalent damping vs. ductility




(d) Design Displacement Spectra

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Select target displacement, $\Delta_d$

Strain, Drift, or Ductility

Calculate yield displacement, $\Delta_y$

Fundamental member property

Calculate equivalent viscous damping, $\zeta$

Relationships between damping and ductility available and easily obtained

Calculate effective period, $T_{eff}$

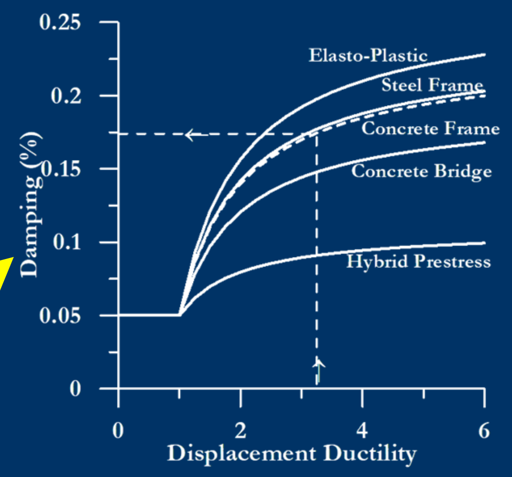
From response spectra

Calculate effective stiffness, $K_{eff}$

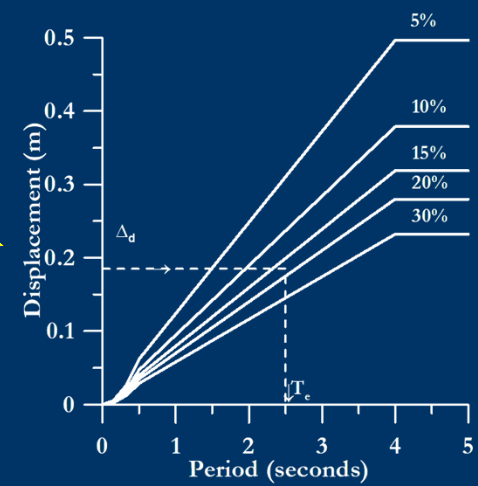
$K_{eff} = 4\pi^2 m/T_{eff}^2$

Calculate design base shear force, $V_b$

$V_b = K_{eff}\Delta_d$




(c) Equivalent damping vs. ductility




(d) Design Displacement Spectra

# DESIGN DISPLACEMENT FOR S.D.O.F STRUCTURES

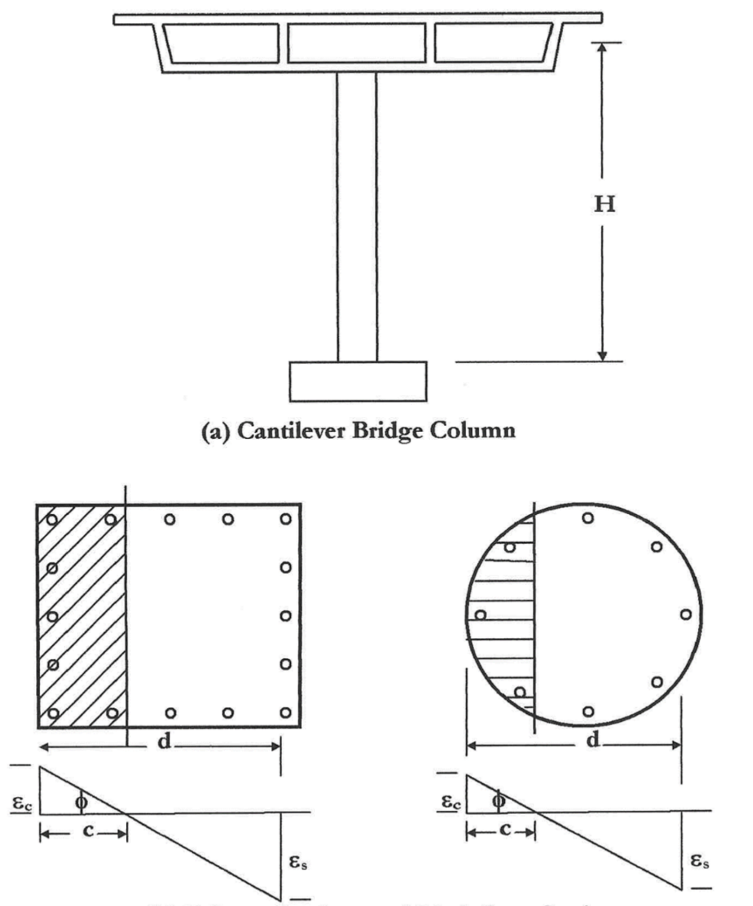
Depends on Design Limit State

Structural displacement limit:
Strain ($\varepsilon$) related, Ductility ($\mu$) related

Non-structural displacement limit:
Drift ($\theta$) related

Chose critical of structural and non-structural limit displacements

# EXAMPLE OF STRAIN LIMIT STATES




(a) Cantilever Bridge Column

(b) Column Sections and Limit State Strains

**Curvature from concrete compression:**

$$\phi_{mc} = \varepsilon_{cm}/c$$

**Curvature from reinforcement tension:**

$$\phi_{ms} = \varepsilon_{sm}/(d-c)$$

**Chose lesser of $\phi_{mc}$ and $\phi_{ms}$**

$$c/D = 0.2 + 0.65P/(f'_{ce}A_g)$$

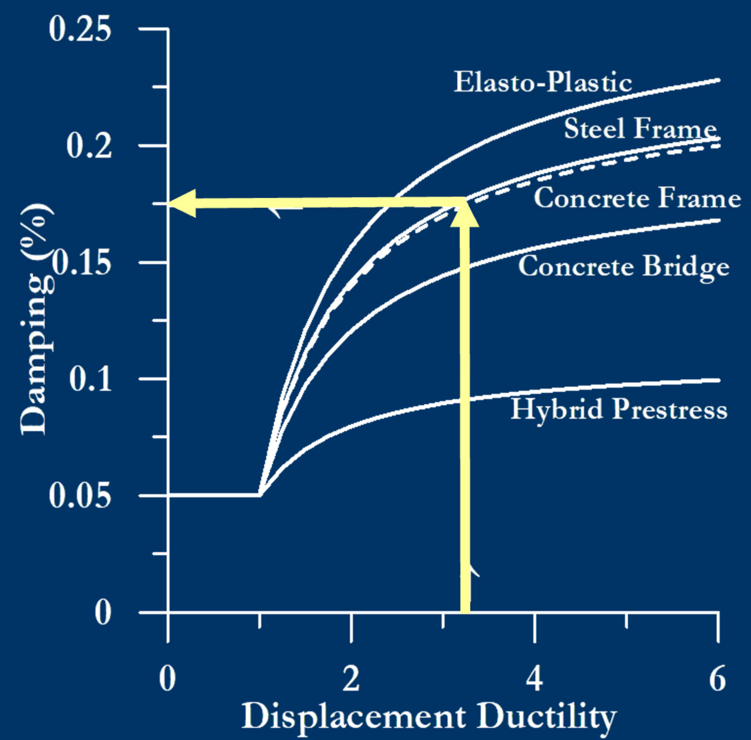
**Design Displacement is:**

$$\Delta_{ds} = \Delta_y + \Delta_p$$
$$= \phi_y H^2/3 + (\phi_m - \phi_y)L_p H$$

$L_p$ = **plastic hinge length**

# Equivalent Viscous Damping

- Based on modifications to Jacobsen's approach (1930), the Equivalent viscous damping can be easily calculated for different hysteretic systems and combined with the elastic damping
- Current work by Brodbeck and Palma is showing that Gulkan and Sozen's substitute damping agrees well with the Modified Jacobsen Approach



# Relationships for Tangent-stiffness Damping, $T_e > 1$ Sec
(include 5% Tangent Stiffness Proportional Damping)

Concrete Wall Building, Bridges (TT): $\xi_{eq} = 0.05 + 0.444\left(\frac{\mu-1}{\mu\pi}\right)$

Concrete Frame Building (TF): $\xi_{eq} = 0.05 + 0.565\left(\frac{\mu-1}{\mu\pi}\right)$

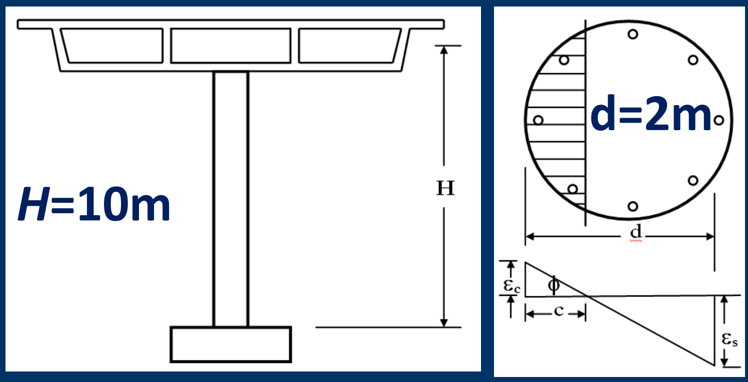
Steel Frame Building (RO): $\xi_{eq} = 0.05 + 0.577\left(\frac{\mu-1}{\mu\pi}\right)$

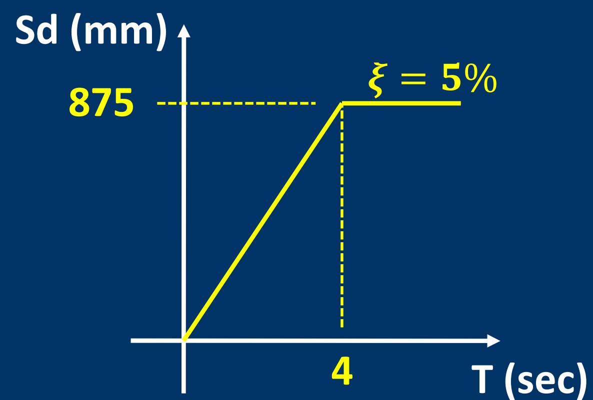
Hybrid Prestressed Frame (FS, $\beta$=0.35): $\xi_{eq} = 0.05 + 0.186\left(\frac{\mu-1}{\mu\pi}\right)$

Friction Slider (EPP): $\xi_{eq} = 0.05 + 0.670\left(\frac{\mu-1}{\mu\pi}\right)$

Bilinear Isolation System (BI, $r$=0.2): $\xi_{eq} = 0.05 + 0.519\left(\frac{\mu-1}{\mu\pi}\right)$

# Example – DDBD SDOF




Displacement Spectra

$$f_y = 470MPa$$
$$E_s = 200GPa$$
$$W = 5000kN$$
$$\theta_d = 0.035$$
$$\mu_d = 4$$

***Target Displacement***

**Drift:** $\Delta_d = (0.035)(10m) = 0.350m$

**Ductility:** $\Delta_d = \mu_d \Delta_y$

$$\Delta_y = \phi_y \frac{H^2}{3}$$
$$\phi_y = 2.25ey/D = 0.00264\ 1/m$$
$$\Delta_y = 0.088\ m$$
$$\Delta_d = 4(0.088) = 0.353\ m$$

# Example – DDBD SDOF

***Equivalent Viscous Damping***

Concrete Wall Building, Bridges (TT): $\xi_{eq} = 0.05 + 0.444\left(\frac{\mu - 1}{\mu\pi}\right)$

Concrete Frame Building (TF): $\xi_{eq} = 0.05 + 0.565\left(\frac{\mu - 1}{\mu\pi}\right)$

Steel Frame Building (RO): $\xi_{eq} = 0.05 + 0.577\left(\frac{\mu - 1}{\mu\pi}\right)$

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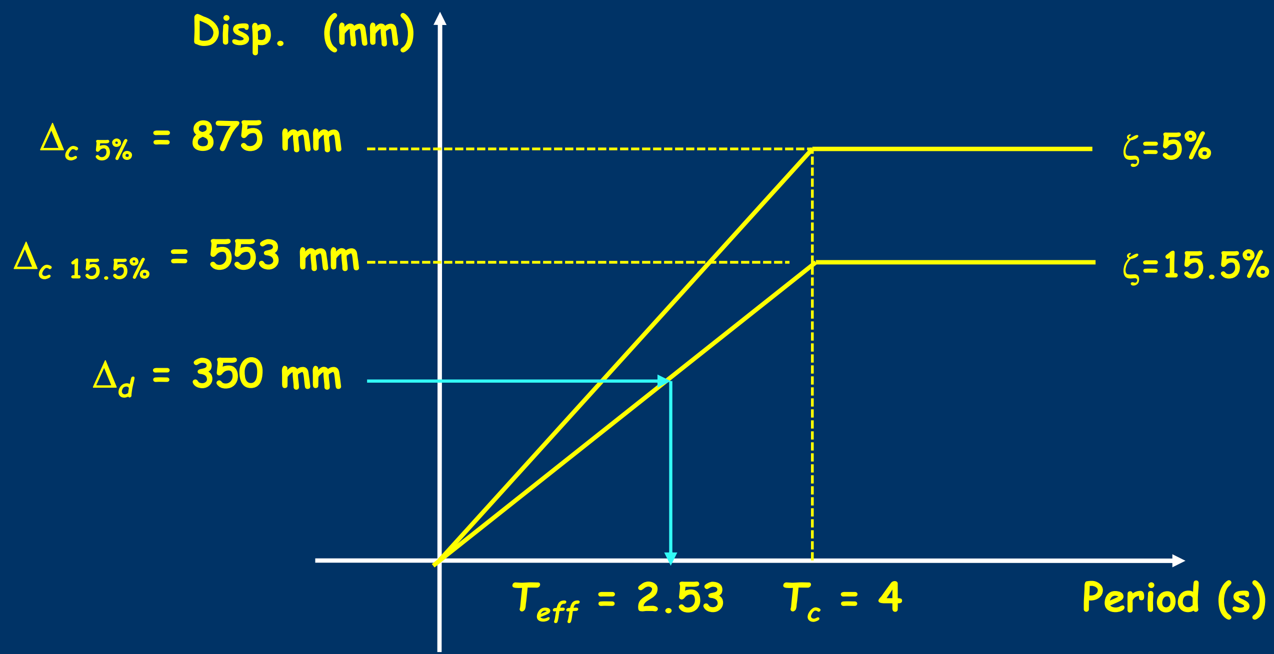
Bilinear Isolation System (BI, $r$ =0.2): $\xi_{eq} = 0.05 + 0.519\left(\frac{\mu - 1}{\mu\pi}\right)$

$$\xi_e = 0.05+0.444(3.97\text{-}1)/3.97\pi = 0.155 \quad (15.5\%)$$

**These expressions all assume 5% tangent stiffness proportional viscous damping and hysteretic damping**

# Example – DDBD SDOF

## *Obtaining Effective Period*




**Note:** $\Delta_{c_{X\%}} = \Delta_{c_{5\%}} \,.\, R_{\xi}$ $\quad R_{\xi} = \sqrt{\dfrac{7}{2+\xi}}$

# Example – DDBD SDOF

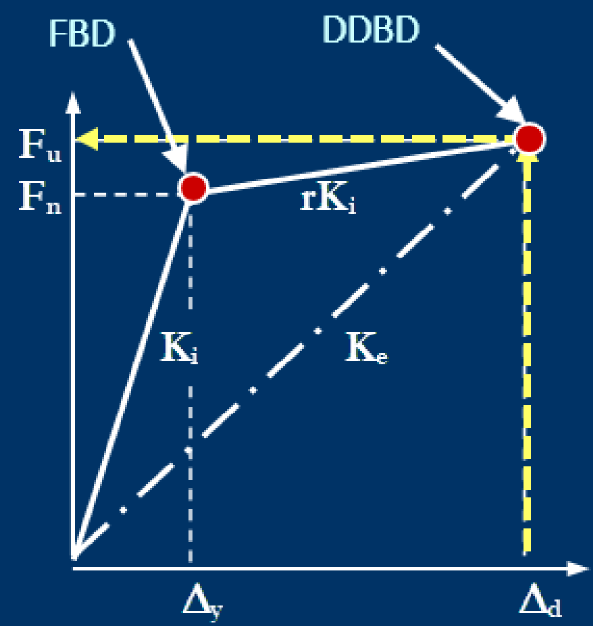
***Obtaining Effective Stiffness*:**

$$K_e = \frac{4\pi^2 m_e}{T_e^2} = \frac{4\pi^2(5000)}{9.805 \times (2.5^3)^2} = 3{,}145\,\frac{kN}{m}$$

***Obtaining Design Base Shear*:**

$$V_{Base} = K_e\Delta_e = 3{,}145\,\frac{kN}{m} \times 0.350\,m$$

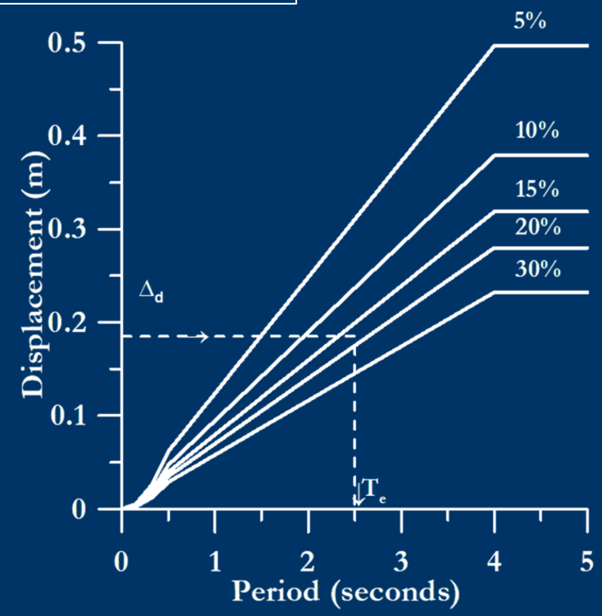
$$V_{Base} = 1{,}100\,kN$$



# Simplified Base Shear Equation for DDBD

$$V_{Base} = K_e \Delta_d = \frac{4\pi^2 m_e}{T_c^2} \times \frac{\Delta_{c,5}^2}{\Delta_d} \times \left(\frac{0.07}{0.02 + \xi}\right)^{2\alpha}$$

$\alpha$ = 0.5 for regular conditions
$\alpha$ = 0.25 for velocity pulse conditions



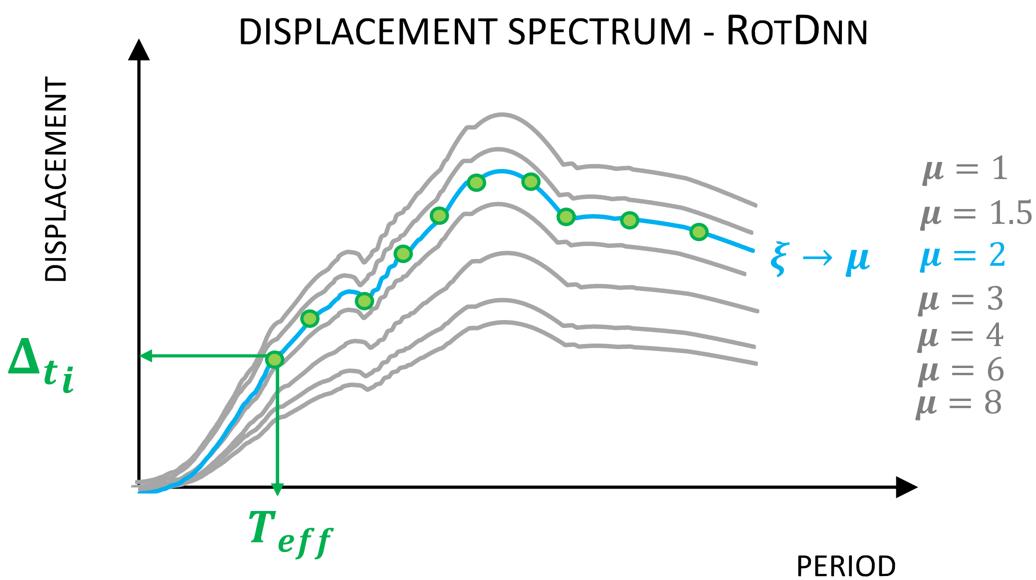
(d) Design Displacement Spectra

**NOTE:**
Damping expressed as ratio in the equation (not %).

Equation assumes a linear DRS to the corner point.

# Bidirectional DDBD (from Ariadne Palma)

For each ground motion pair:




$$T_{eff} \rightarrow k_{eff} \rightarrow \mathrm{V_b} \rightarrow \mathrm{M_N} \rightarrow \mathrm{M_y} \rightarrow \mathrm{I_{CR}}$$

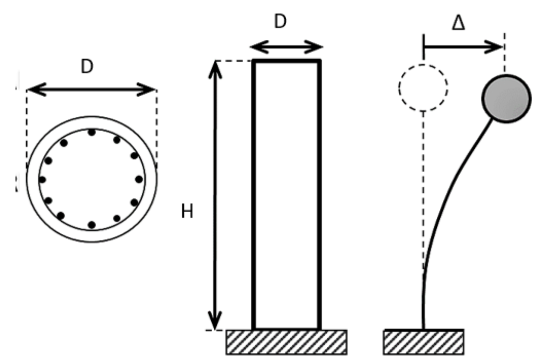
**For all designs:**

RC circular columns in single bending

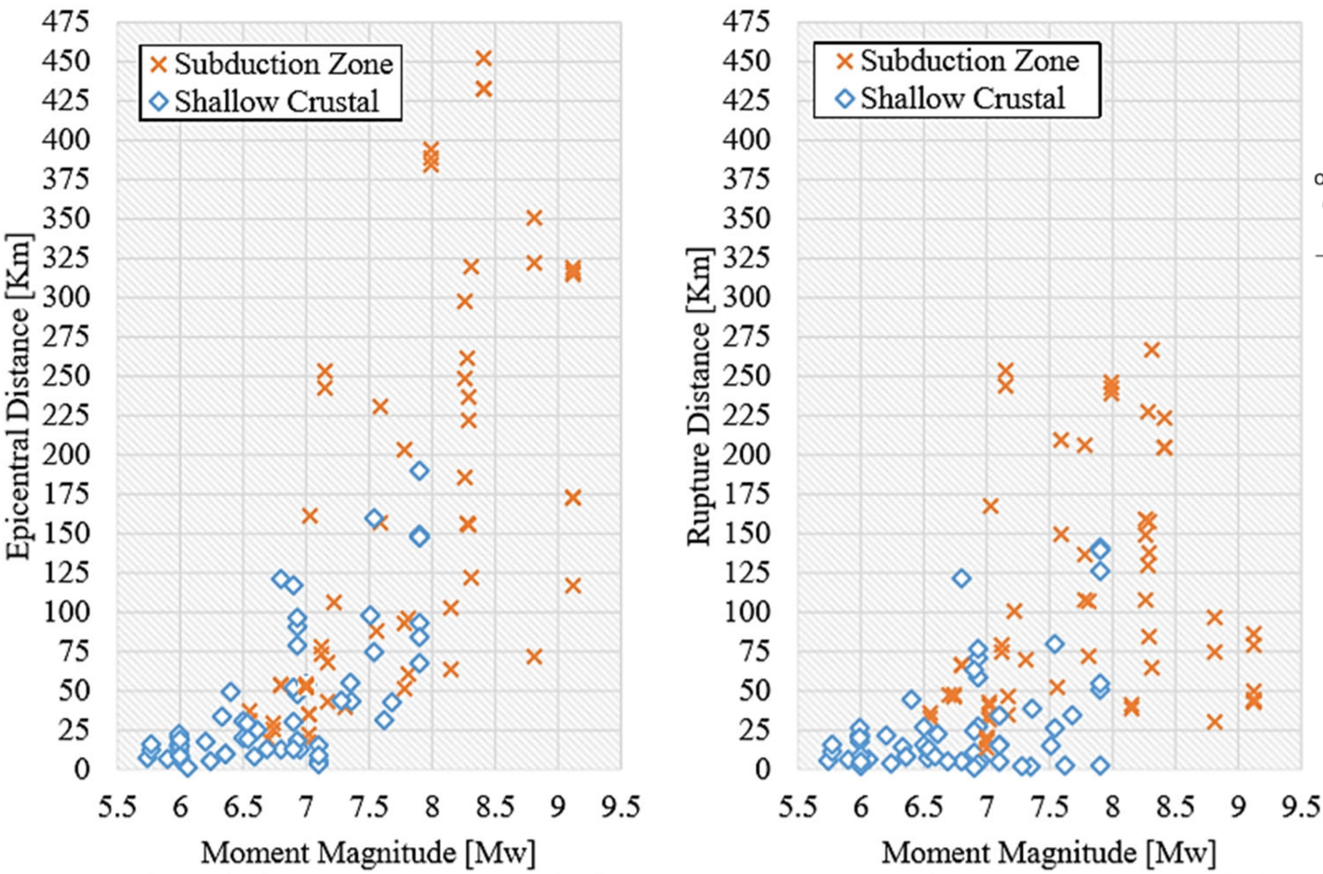
Constant material properties

Constant height

Force-Displacement bilinear factor ($r_{\Delta}$) equal to **5%**.

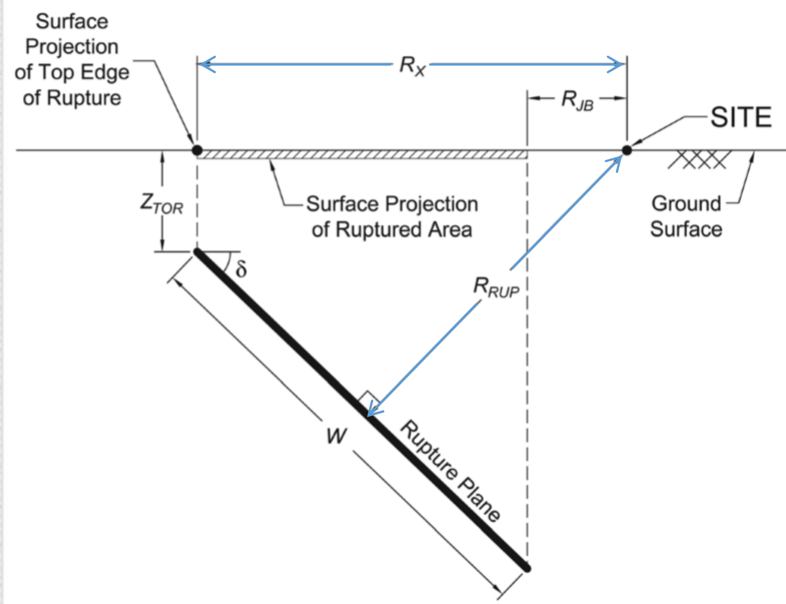


Data distribution for the selected 120 as-recorded pairs.




(a) Epicentral Distance vs Moment Magnitude

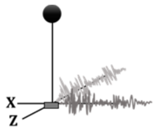
(b) Rupture Distance vs Moment Magnitude




$R_x$: *Epicentral Distance*

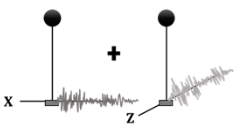
$R_{rup}$: *Rupture Distance*

# Displacement ratios - $f(T_{eff})$



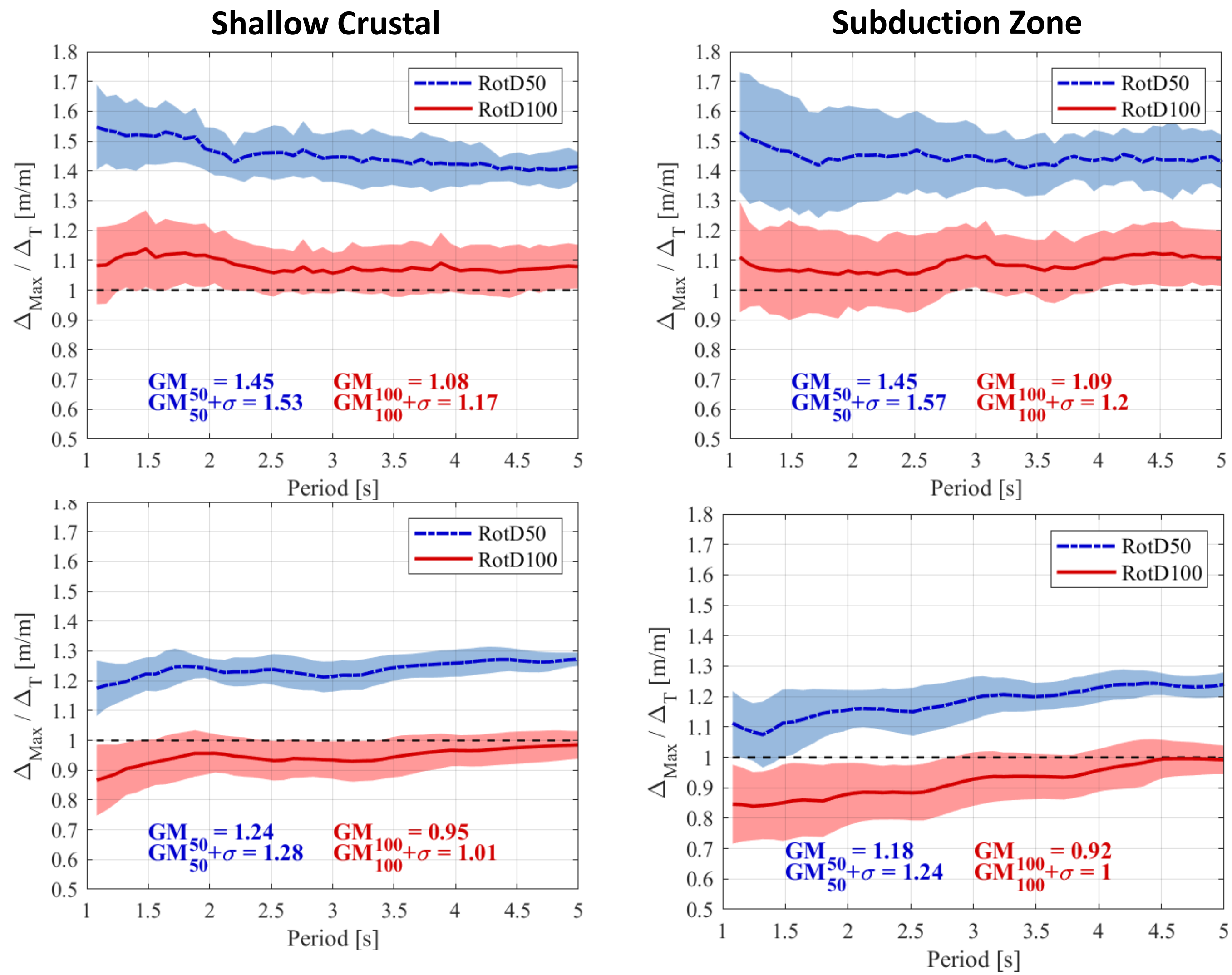
**Coupled Response**

| *nn* | 50 | 100 |
|---|---|---|
| *Factor* | 1.45 | 1.08 |
| Upper (+ σ) | 1.53 | 1.17 |
| ***nn*** | **50** | **100** |
| *Factor* | 1.45 | 1.09 |
| Upper (+ σ) | 1.57 | 1.20 |



**Uncoupled Response**

| *nn* | 50 | 100 |
|---|---|---|
| *Factor* | 1.24 | 0.94 |
| Upper (+ σ) | 1.28 | 1.01 |
| ***nn*** | **50** | **100** |
| *Factor* | 1.18 | 0.92 |
| Upper (+ σ) | 1.24 | 1.00 |



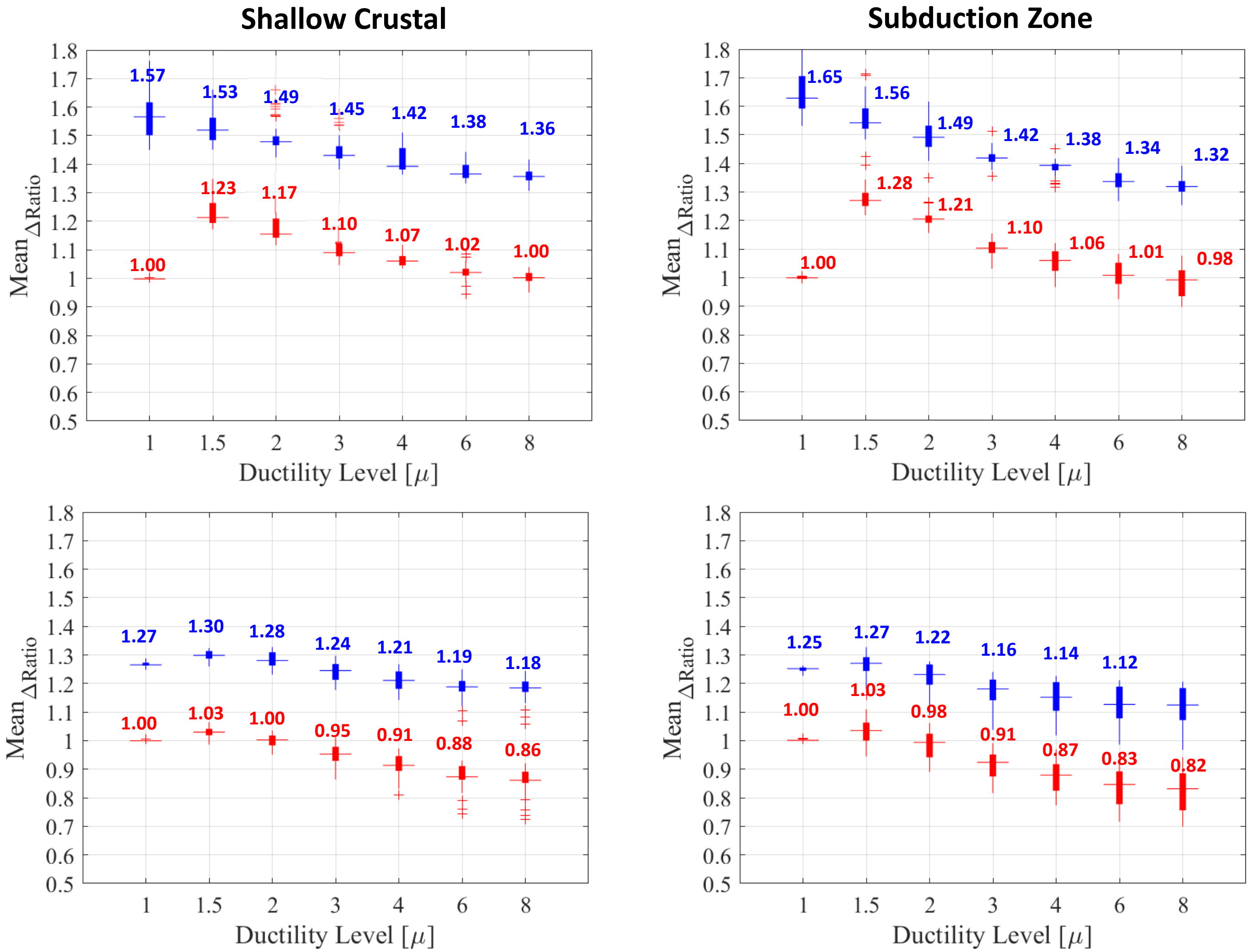
# Displacement ratios - $f(\mu_\Delta)$

**Coupled Response**

| ***nn*** | **50** | **100** |
|---|---|---|
| *Factor* | 1.45 | 1.08 |
| Upper (+ σ) | 1.53 | 1.17 |
| ***nn*** | **50** | **100** |
| *Factor* | 1.45 | 1.09 |
| Upper (+ σ) | 1.57 | 1.20 |

**Uncoupled Response**

| ***nn*** | **50** | **100** |
|---|---|---|
| *Factor* | 1.24 | 0.94 |
| Upper (+ σ) | 1.28 | 1.01 |
| ***nn*** | **50** | **100** |
| *Factor* | 1.18 | 0.92 |
| Upper (+ σ) | 1.24 | 1.00 |



# DDBD for Multi-span Bridges

# The DDBD Steps for MDOF Systems

1. ***Select Target Displaced Shape***
2. Calculate system displacement
3. ***Calculate system damping***
4. Calculate effective period
5. Calculate effective stiffness
6. Calculate base shear
7. Distribute base shear & ***Structural analysis***
8. ***Design and detail***

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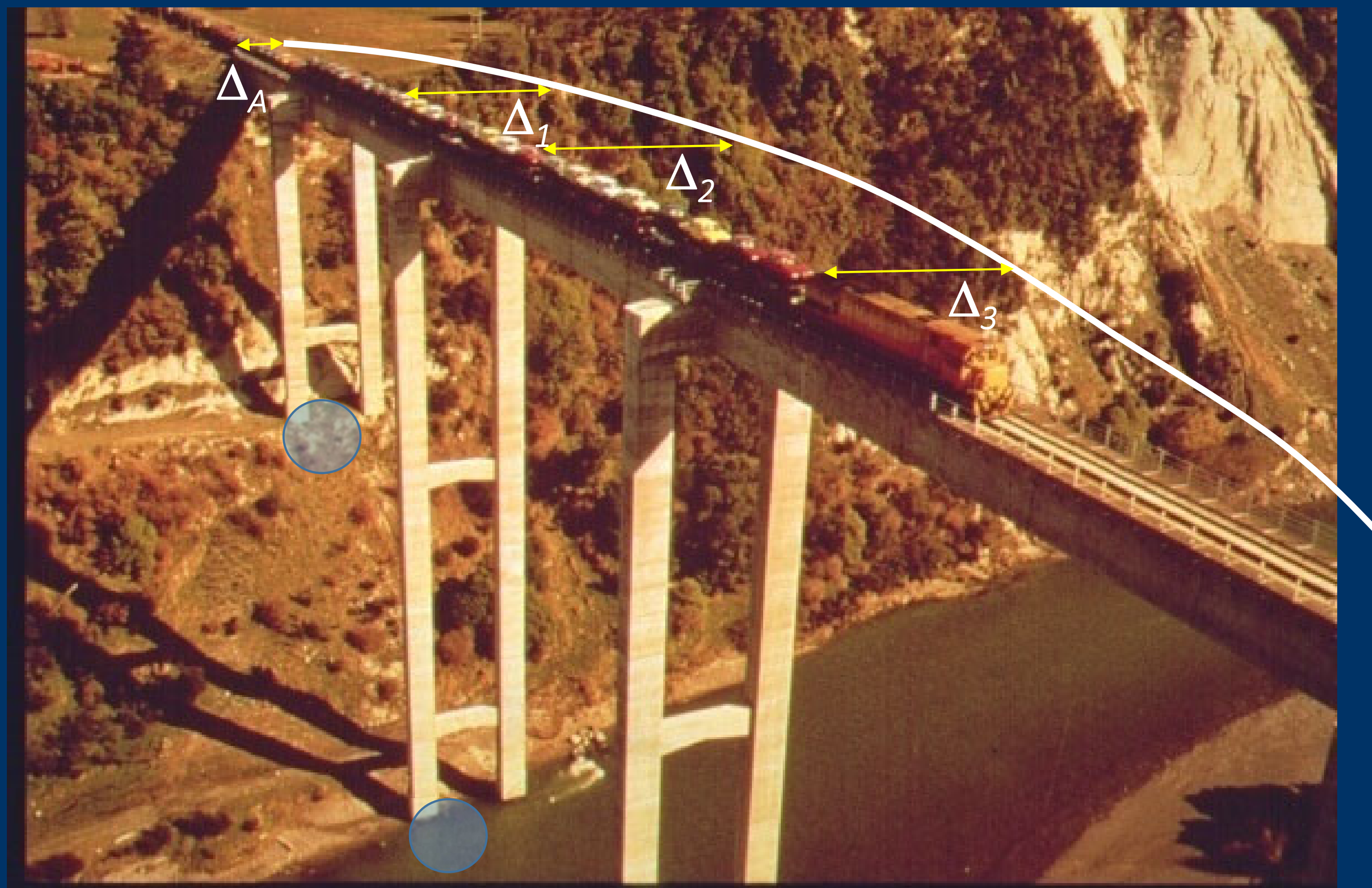
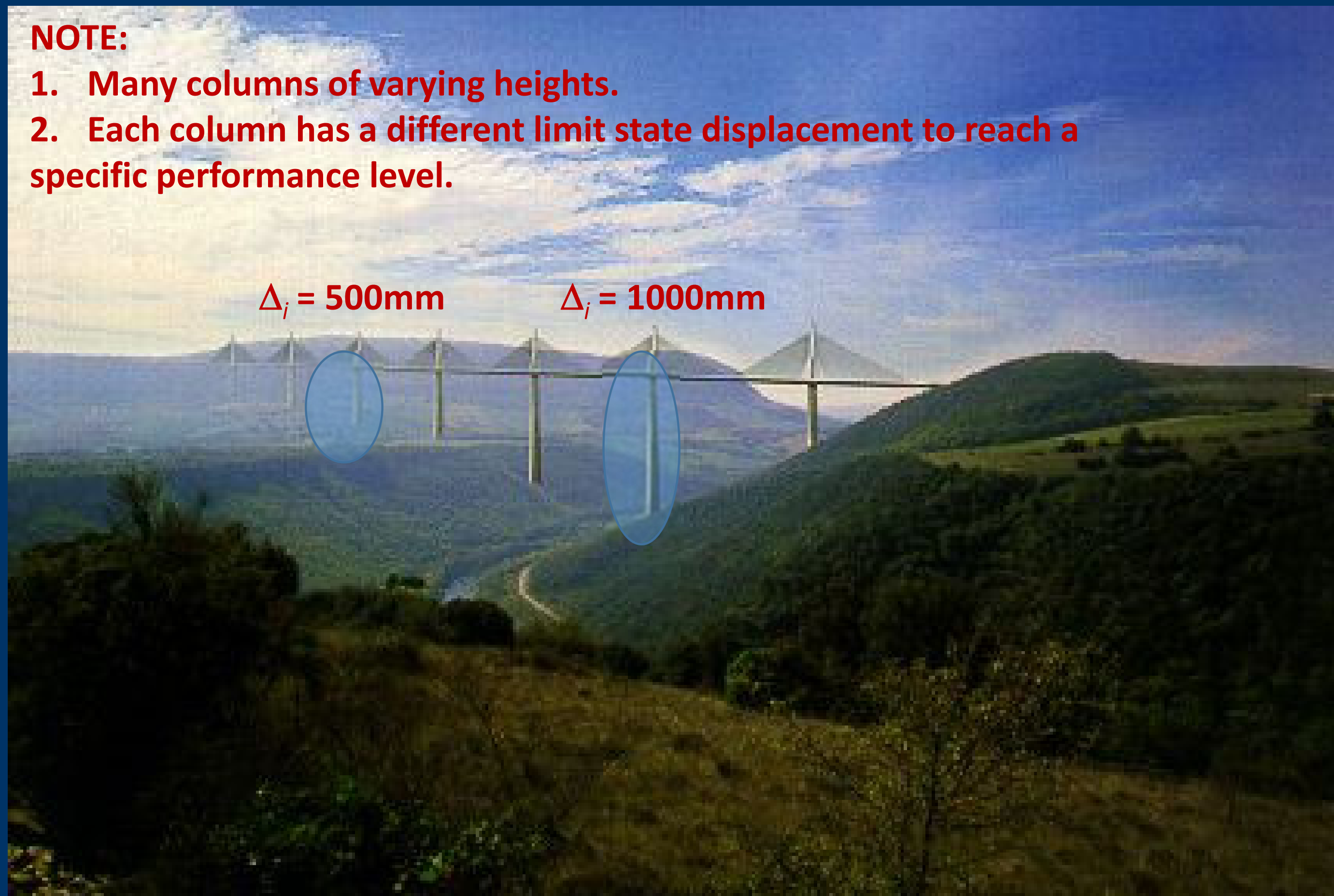
# What’s different for a multispan bridge?

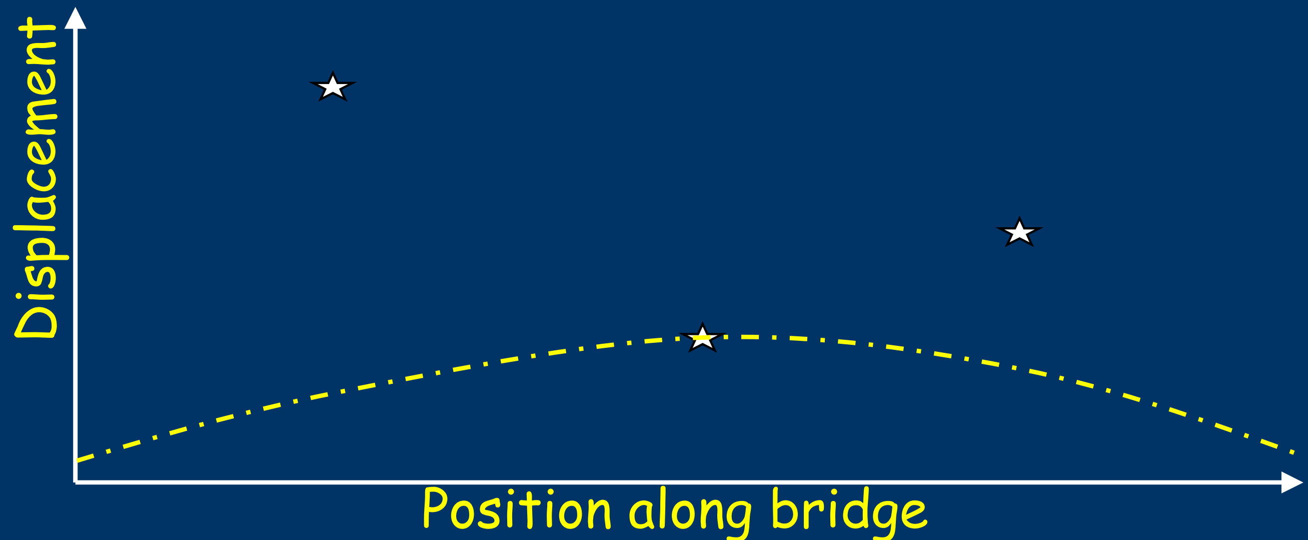


**NOTE:**

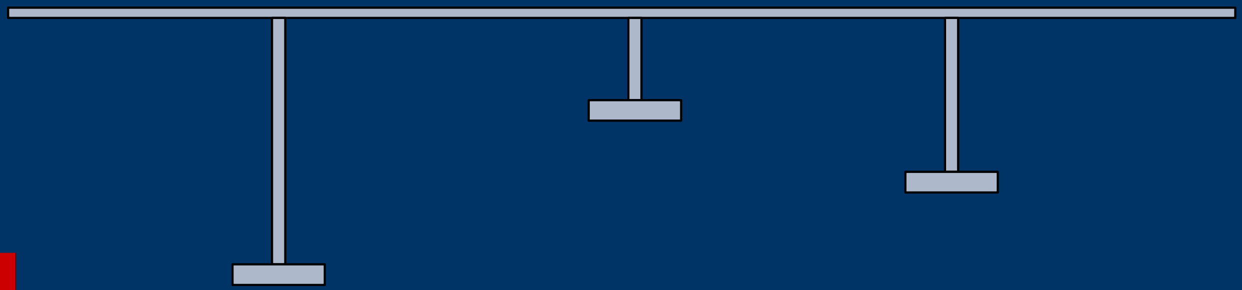
1. **Many columns of varying heights.**
2. **Each column has a different limit state displacement to reach a specific performance level.**



# Obtaining Transverse Displaced Shape




Note: Stars are limit state displacements based on strain, ductility, or drift



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# System Displacement and Effective Mass

From **work balance** between MDOF and SDOF systems:

$$\Delta_d = \sum_{i=1}^{n}\left(m_i \Delta_i^2\right) / \sum_{i=1}^{n}\left(m_i \Delta_i\right)$$

From **force equilibrium** between MDOF and SDOF systems:

$$m_e = \sum_{i=1}^{n}\left(m_i \Delta_i\right) / \Delta_d$$

# Lateral Force Distribution

$$F_i = V_B\left(m_i\Delta_i\right) / \sum_{i=1}^{n}\left(m_i\Delta_i\right)$$

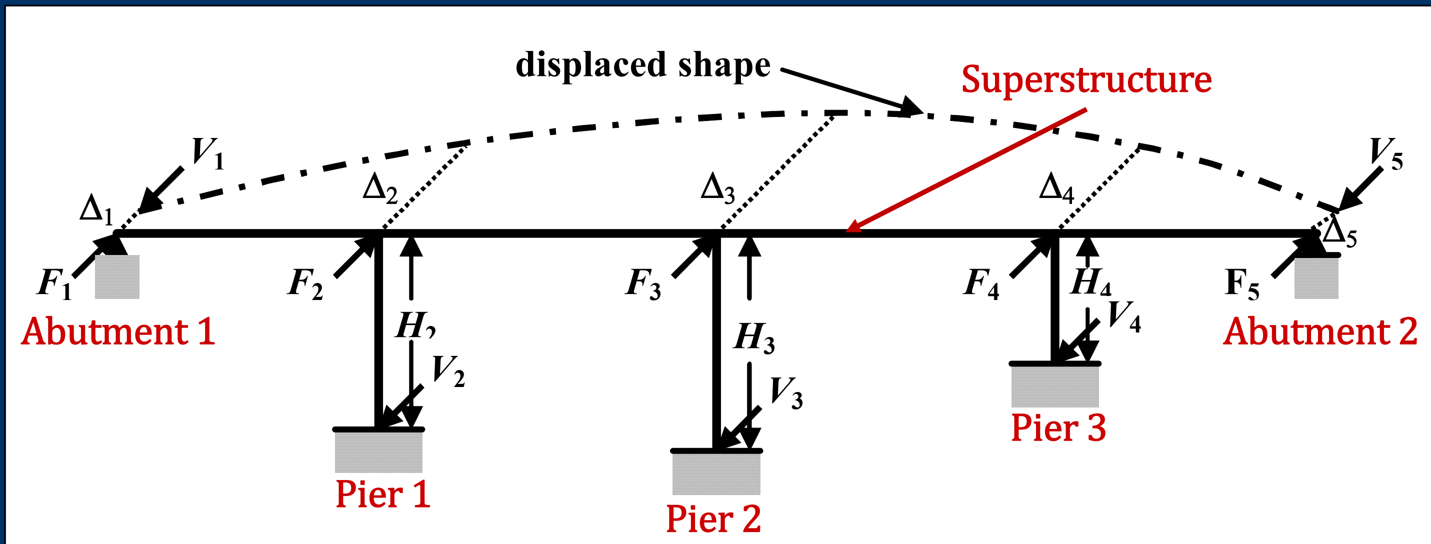
Force is distributed in proportion to mass and pier top displacement.

# Bridge components that do work – transverse direction

Abutments

Column
Plastic
Hinges

Superstructure




$V$ = Shear Force
$F$ = Inertia Force
$\Delta$ = Lateral Displacement

Pier Damping: $\xi = 0.05 + 0.444\left(\frac{\mu - 1}{\mu\pi}\right)$ where $\mu_i = \Delta_i / \Delta_{yi}$

Superstructure Damping: 5% (elastic)

Abutment Damping: varies – depends on soil

System Damping:

$$\xi_e = \frac{F_{SS}\Delta_{SS}\xi_{SS} + F_a\Delta_a\xi_a + \sum V_i.\Delta_i\,\xi_i}{F_{SS}\Delta_{SS} + F_a\Delta_a + \sum V_i.\Delta_i}$$

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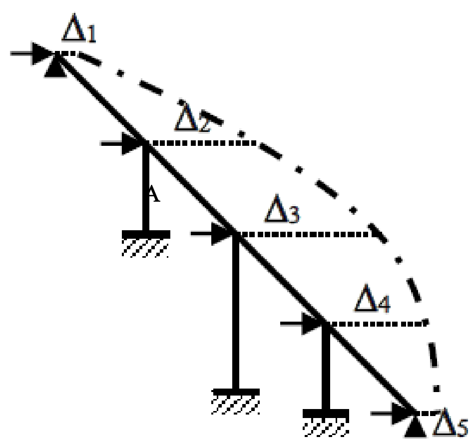
# DDBD Steps for Multi-Span Bridge – Transverse

1. Initial guess of fraction of lateral force carried by superstructure bending.

$$x = ?$$

2. Displaced shape of the bridge.




(d) Symm,. Restrained at abu
Flexible SS

Displaced Shape → Initial Guess

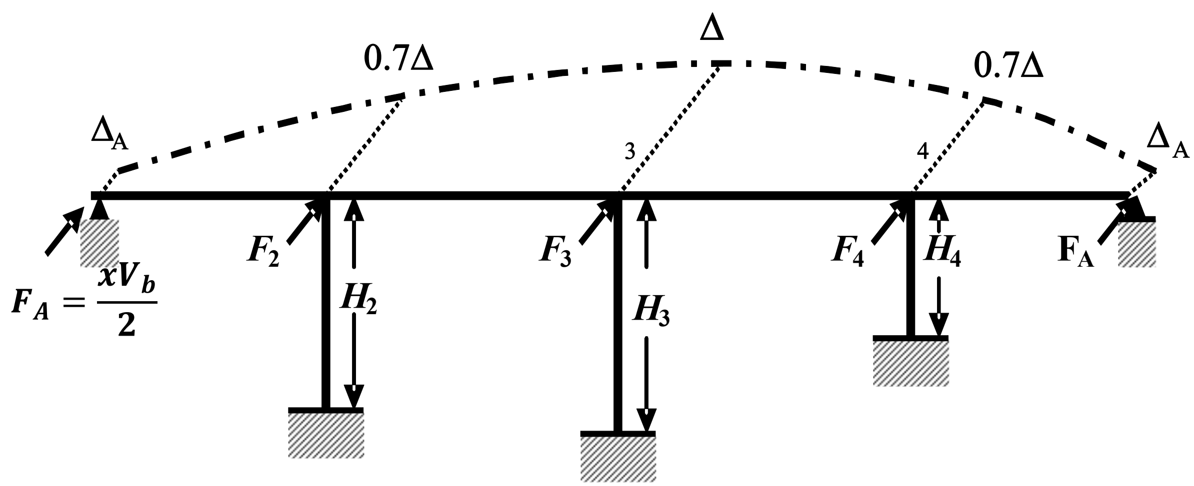



Fig.10.16 Different Possible Transverse Displacement Profiles for Bridges

# DDBD Steps for Multi-Span Bridge – Transverse

## 2. Displaced shape of the bridge

- Pier Limit State Displacement:

$$\Delta_d = \Delta_y + (\phi_{ls} - \phi_y) L_p H$$

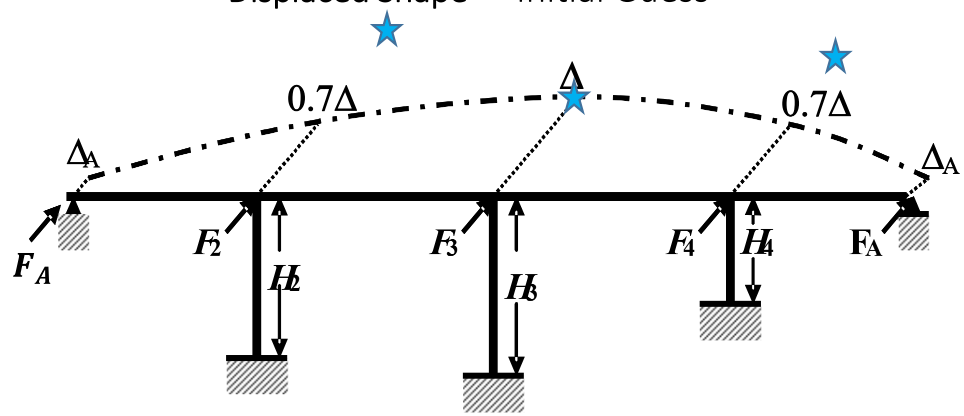
WHERE

Plastic Hinge Length: Maximum of $L_p = kL_c + 0.022F_{ye}.d_{bl}$

$$L_p = 0.044F_{ye}.d_{bl}$$

- Initial Displacement Profile: Initial Guess

Displaced Shape → Initial Guess



# DDBD Steps for Multi-Span Bridge – Transverse

3. SDOF System Displacement:

$$\Delta_{d_{SYS}} = \sum_{i=1}^{n}(m_i\Delta_i^2) \,/\, \sum_{i=1}^{n}(m_i\Delta_i)$$

WHERE

$$m_i = L_{trib_i} \times (W_S)$$

$$L_{trib} = Tributary\ length\ carried\ by\ each\ pier, i$$
$$W_s = Superstructure\ weight\ per\ linear\ unit$$

4. System Effective mass:

$$m_{eff} = \sum_{i=1}^{n}(m_i\Delta_i) \,/(\Delta_{d_{SYS}})$$

# DDBD Steps for Multi-Span Bridge – Transverse

5. Ductility and equivalent damping:

- Pier Displacement Ductility:

$$\mu_{\Delta_i} = \Delta_i / \Delta_y$$

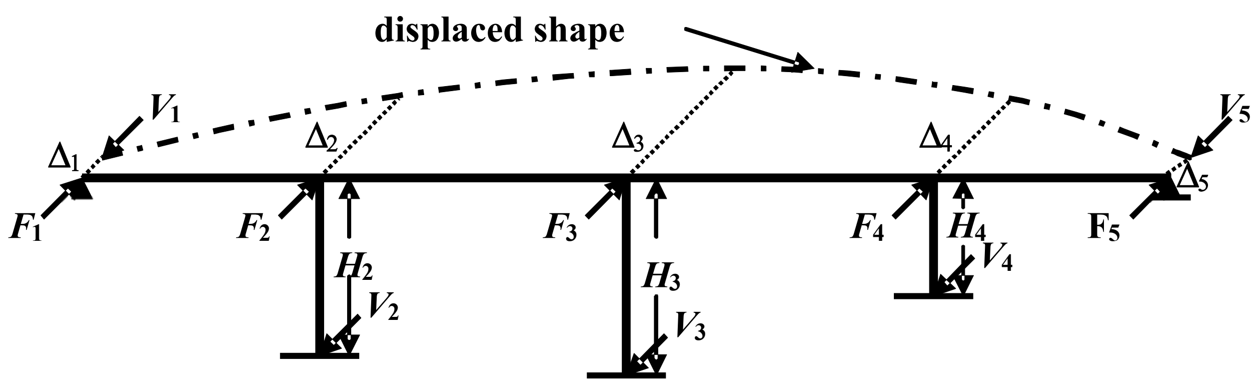
- Pier Equivalent Damping:

$$\xi_i = 0.05 + 0.444\left(\frac{\mu - 1}{\mu\pi}\right)$$

# DDBD Steps for Multi-Span Bridge – Transverse

6. System Damping:

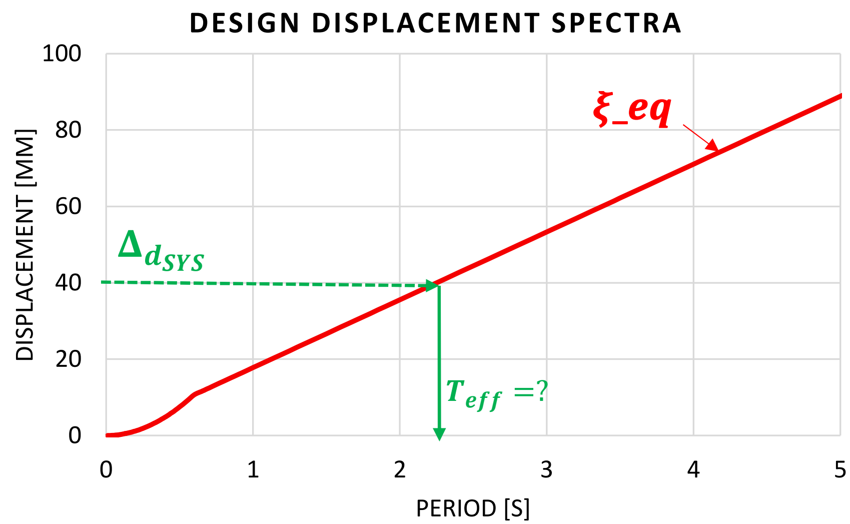
$$\xi_{eq} = \frac{0.05x.\Delta_{d_{SYS}} + (1-x).\dfrac{\left(\sum_{i=2}^{4}\dfrac{1}{H_i}.\Delta_i\xi_i\right)}{\sum_{i=2}^{4}\dfrac{1}{H_i}}}{x.\Delta_{d_{SYS}} + (1-x).\dfrac{\left(\sum_{i=2}^{4}\dfrac{1}{H_i}.\Delta_i\right)}{\sum_{i=2}^{4}\dfrac{1}{H_i}}}$$



# DDBD Steps for Multi-Span Bridge – Transverse

7. System Properties:

- Effective Period (s):




- Effective Stiffness: $k_{eff} = (4\pi^2 m_{eff})/({T_{eff}}^2)$

- Design Base Shear: $V_b = k_{eff} \cdot \Delta_{d_{SYS}}$

# DDBD Steps for Multi-Span Bridge – Transverse

8. Find Lateral Force vector (consistent with assumed displaced shape):

$$F_i = V_b(m_i\Delta_i)/\sum(m_i\Delta_i)$$

9. Structural Analysis:

- Shear force per Pier and Abutments:

$$\mathrm{V_i} = (1-x).\left(\frac{\frac{1}{H_i}}{\sum_{i=2}^{4}\frac{1}{H_i}}\right).V_b = V_{ratio_i}.V_b$$

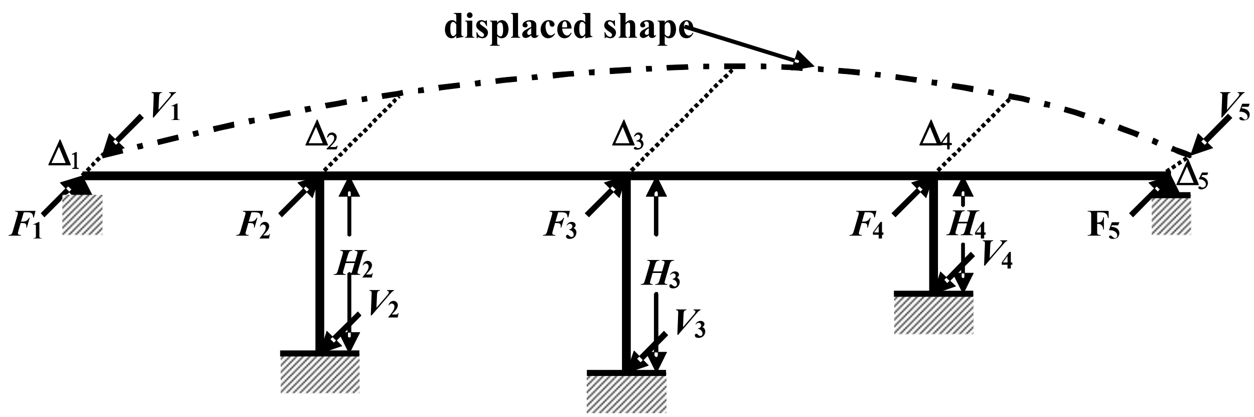
- Pier and Abutment Stiffness:

$$k_{eff_i} = V_i/\Delta_i$$

# DDBD Steps for Multi-Span Bridge – Transverse

9. Structural Analysis:

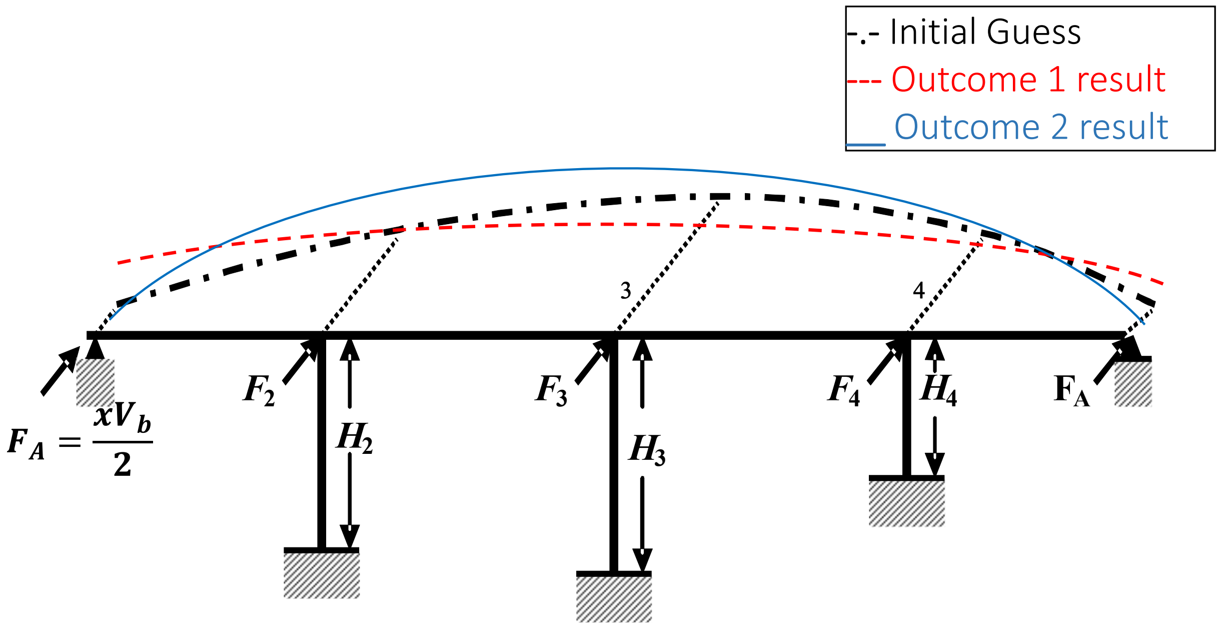
- Shear force per Pier and Abutments: $V_i = (1-x).\left(\frac{\frac{1}{H_i}}{\sum_{i=2}^{4}\frac{1}{H_i}}\right).V_b = V_{ratio}.V_b$
- Pier and Abutment Stiffness: $k_{eff\,i} = V_i/\Delta_i$




*How does the displaced shape compare to the initial guess?*

# DDBD Steps for Multi-Span Bridge – Transverse

9. From Structural Analysis:
   - If the displacement shape does not match the initial guess:




Outcome 1:

Increase guess of x

*Assign more force to abutments.*

Outcome 2:

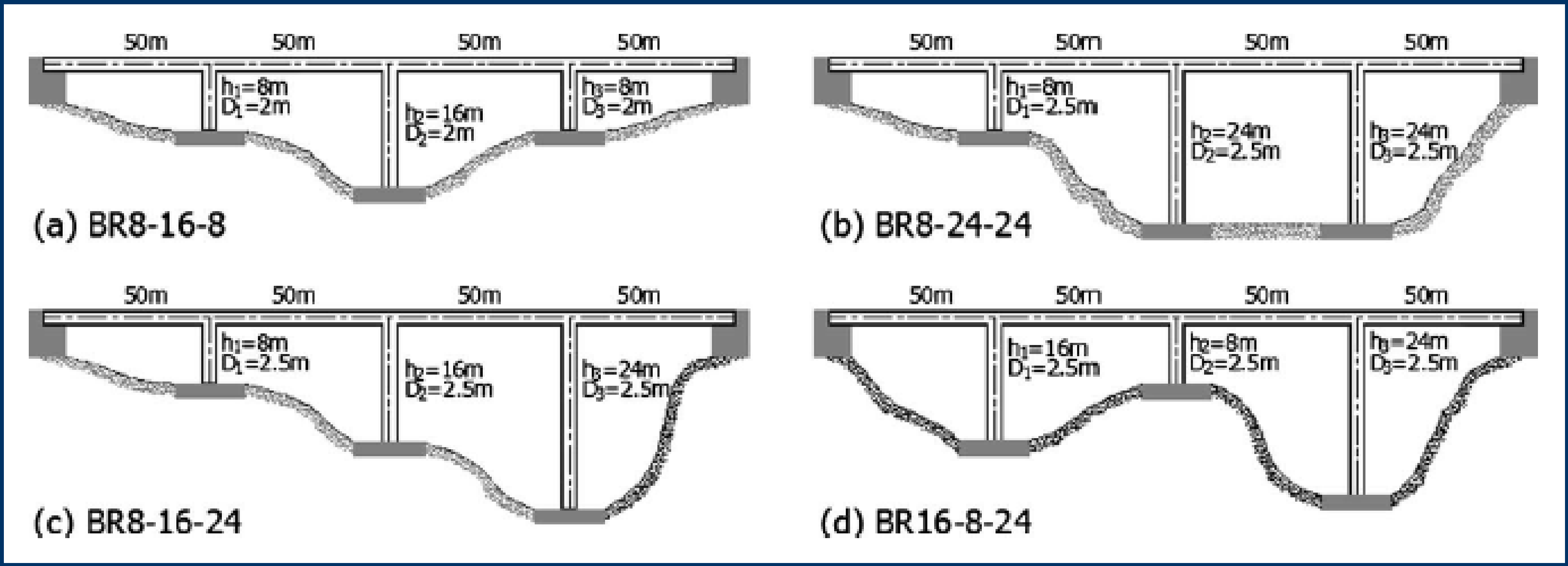
Decrease guess of x

*Assign more force to piers.*

*How does the displaced shape compare to the initial guess?*

# DDBD for Multi-Span Bridge

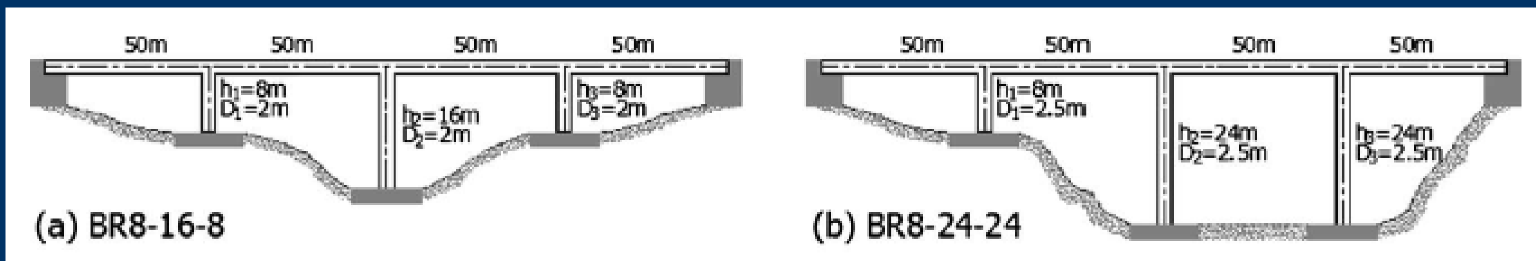
- Four Multi span bridge configurations considered in the study

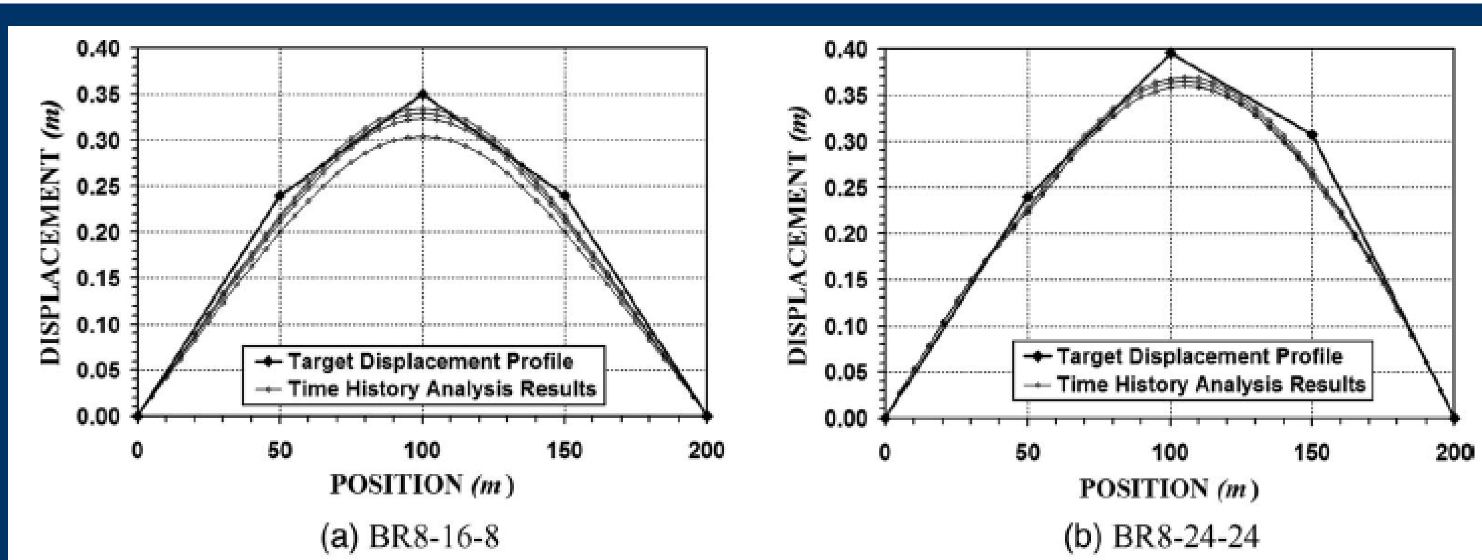



(Dwairi & Kowalsky 2006)

# DDBD MDOF– Modal Analysis: Examples

- Maximum displacements for bridges with flexible patterns

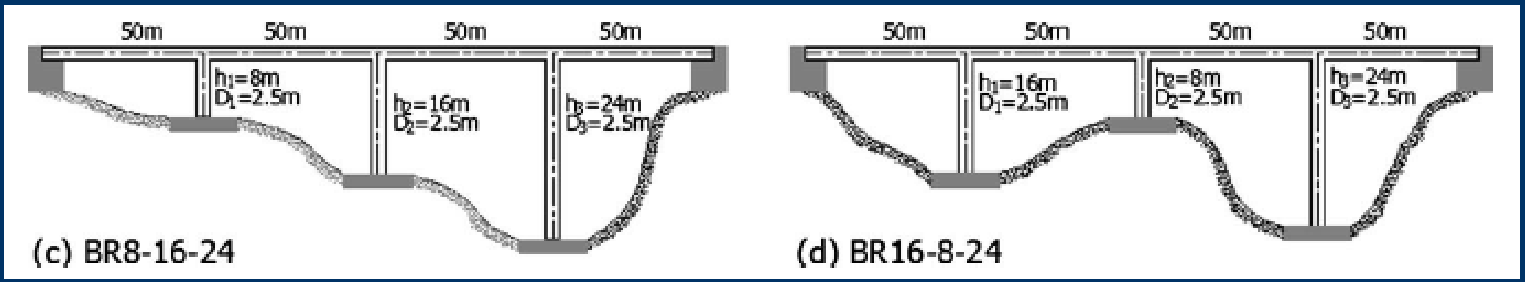


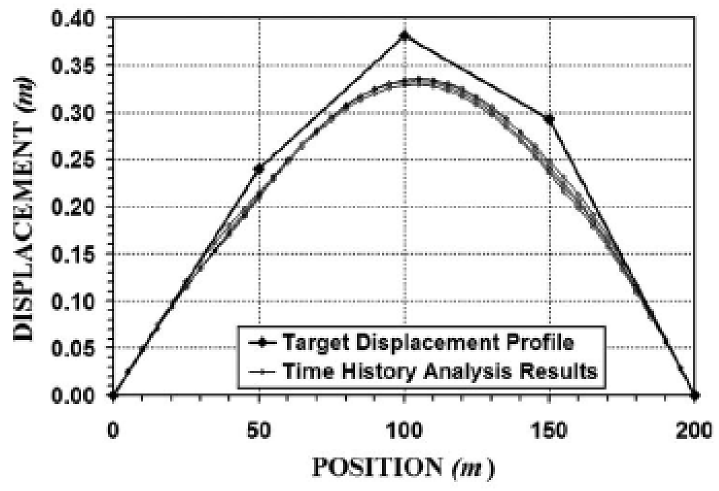




(Dwairi & Kowalsky 2006)

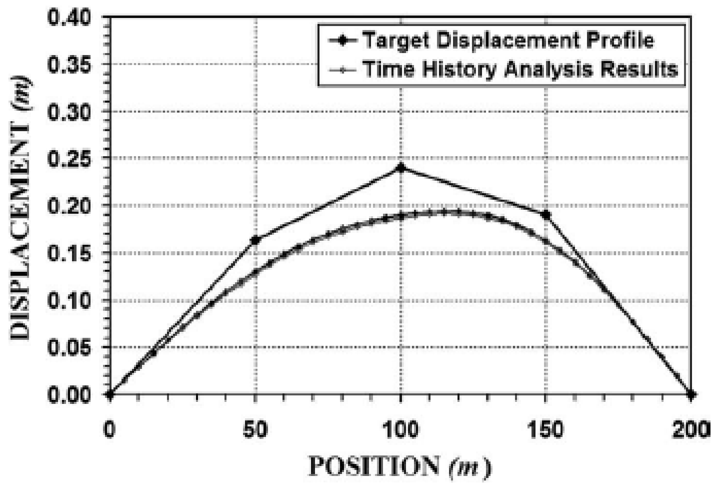
# DDBD MDOF– Modal Analysis: Examples

- Maximum displacements for bridges with flexible patterns



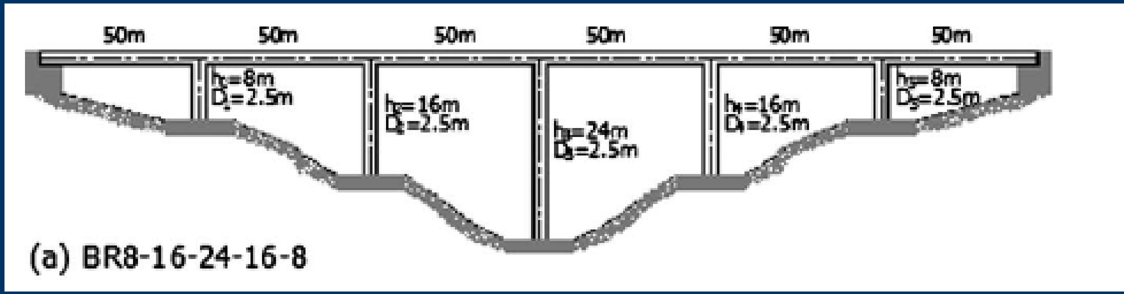

(c) BR8-16-24

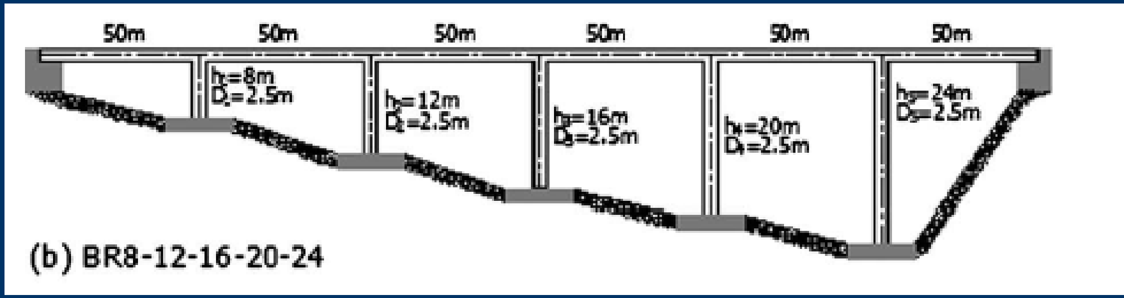



(d) BR16-8-24

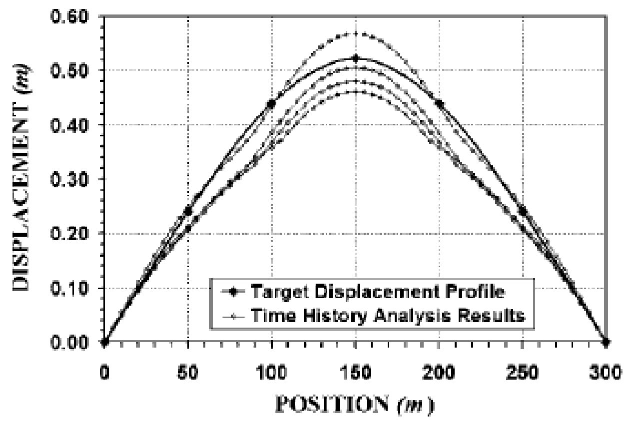
(Dwairi & Kowalsky 2006)

# DDBD MDOF– Modal Analysis: Examples

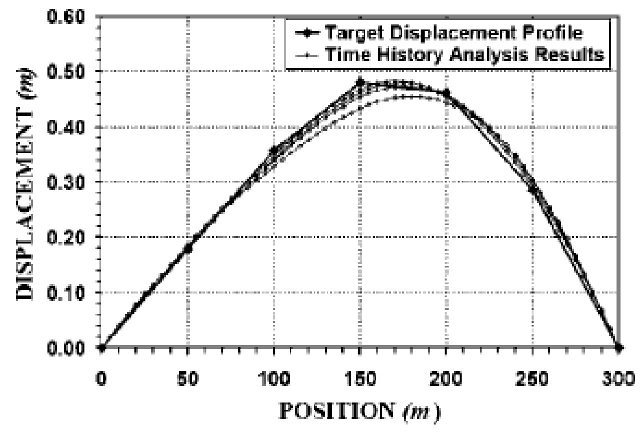



(Dwairi & Kowalsky 2006)

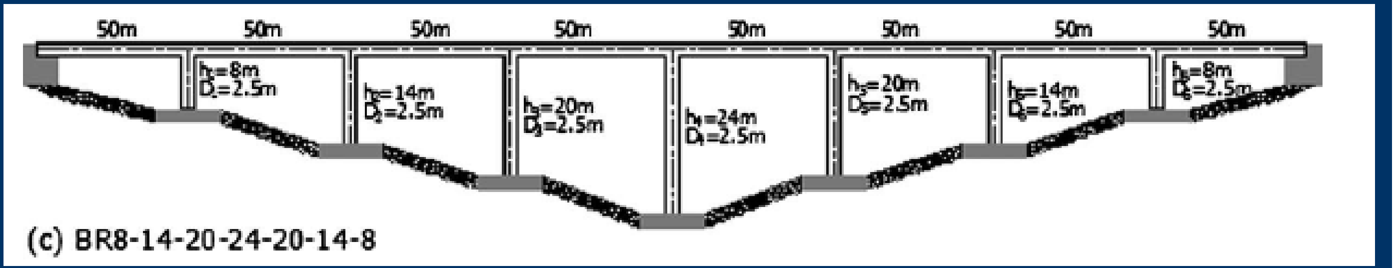


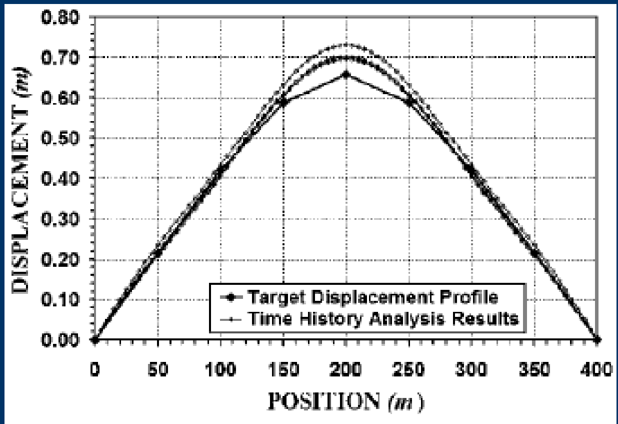

(a) BR 8-16-24-16-8



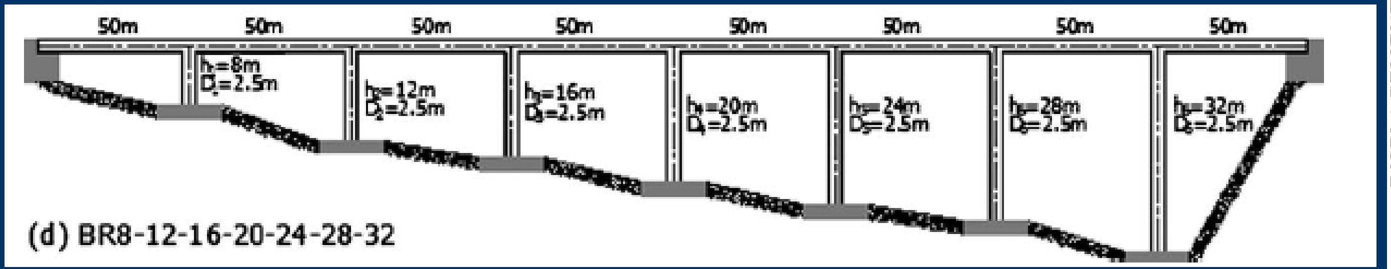

(b) BR 8-12-16-20-24

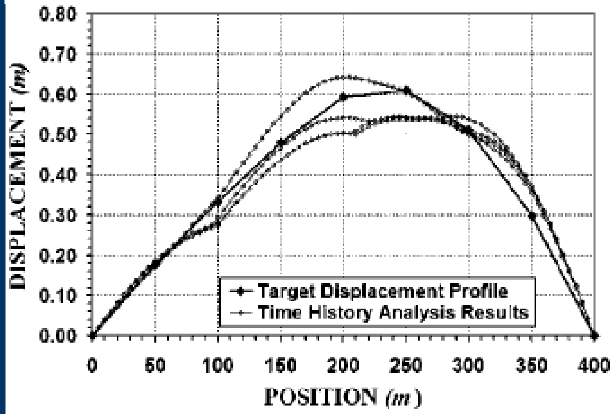
# DDBD MDOF– Modal Analysis: Examples



(c) BR 8-14-20-24-20-14-8



(d) BR 8-12-16-20-24-28-32

(Dwairi & Kowalsky 2006)

# Structural Walls: Items to Consider

- Target displaced shapes
- Damping for walls of unequal length
- All Equivalent MDOF expressions apply
  - System Displacement
  - System Mass
  - Lateral force distribution
- Higher Modes
- Torsion

## CANTILEVER WALL DESIGN PROFILES:

If roof drift is less than the code drift limit $\theta_c$, the design displacement profile is:

$$\Delta_i = \Delta_{yi} + \Delta_{pi} = \frac{\varepsilon_y}{l_w} H_i^2 \left(1 - \frac{H_i}{3H_n}\right) + \left(\phi_m - \frac{2\varepsilon_y}{l_w}\right) L_p H_i$$

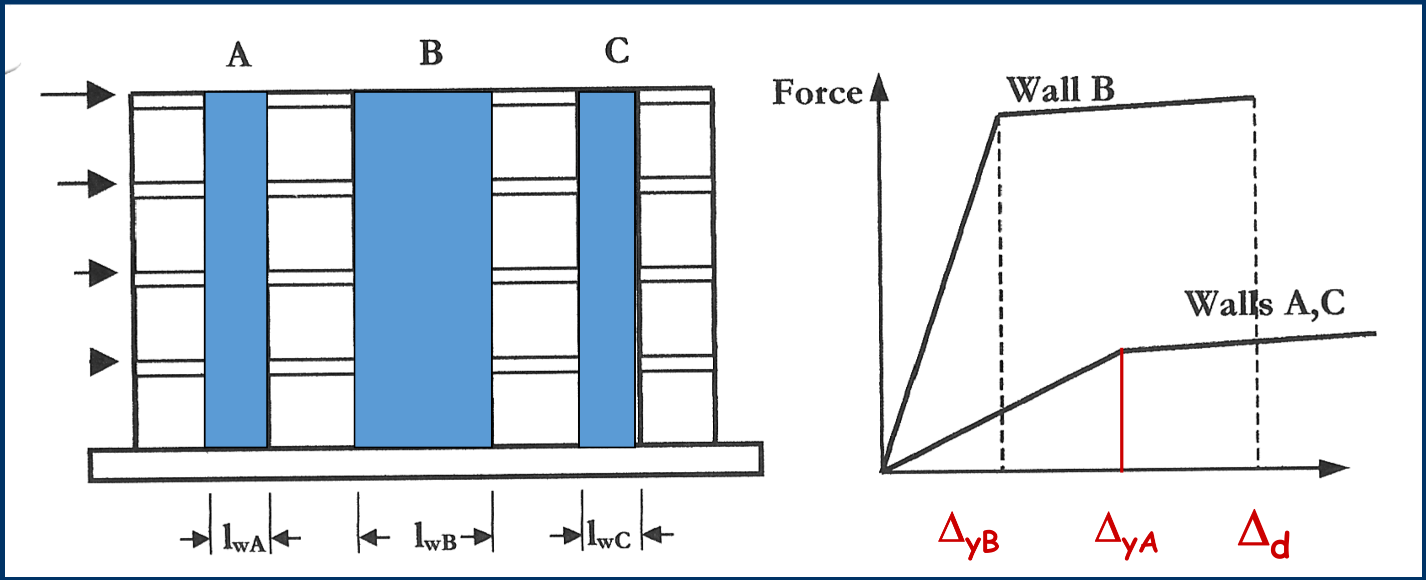
If the code drift limit governs the roof drift, the design displacement profile is:

$$\Delta_i = \Delta_{yi} + \left(\theta_c - \theta_{yn}\right) H_i = \frac{\varepsilon_y}{l_w} H_i^2 \left(1 - \frac{H_i}{3H_n}\right) + \left(\theta_c - \frac{\varepsilon_y H_n}{l_w}\right) H_i$$

Note: Yield curvature for rectangular section walls is

$$\phi_y = 2\varepsilon_y / l_w$$

# DAMPING FOR WALL BUILDING




Walls have different ductilities ($\Delta_d/\Delta_{yA}$, $\Delta_d/\Delta_{yB}$), hence damping.

System Damping:

$$\xi_e = \sum_{j=1}^{m}\left(V_j \xi_j\right) / \sum_{j=1}^{m} V_j$$

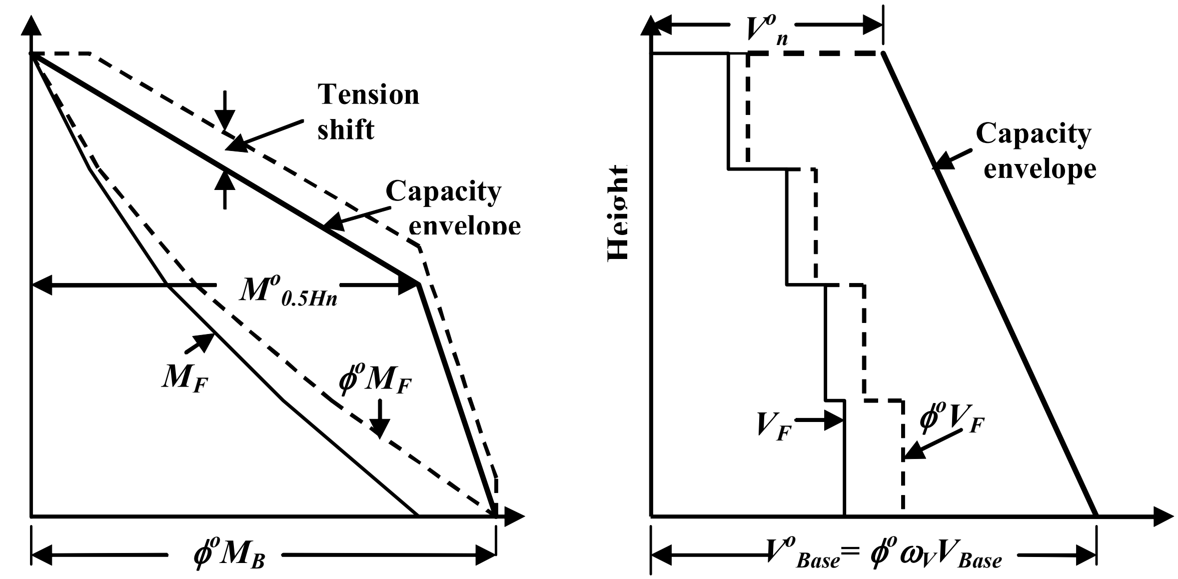
Note: chose wall strength

Proportional to $l_w^2$:

$$\xi_e = \sum_{j=1}^{m}\left(l_{wj}^2 \xi_j\right) / \sum_{j=1}^{m} l_{wj}^2$$

# SIMPLIFIED WALL CAPACITY-DESIGN EQUATIONS

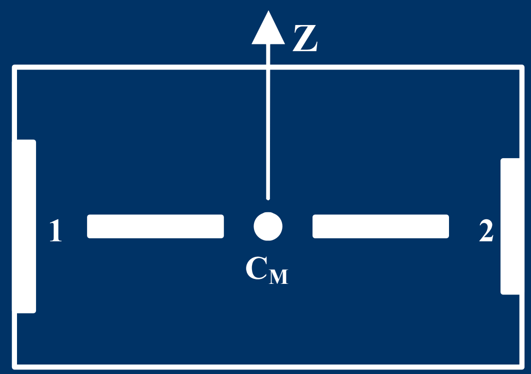
**Moment:** $M^o_{0.5Hn} = C_{1,T} \cdot \phi^o M_B$ **where** $C_{1,T} = 0.4 + 0.075 T_i \left( \frac{\mu}{\phi^o} - 1 \right) \geq 0.4$



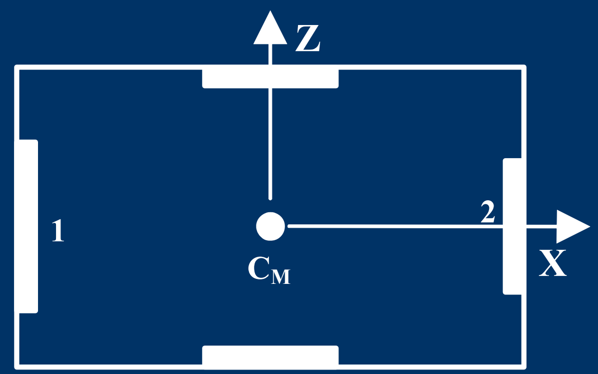

(a) Moment Capacity Envelope (b) Shear Force Capacity

$$V^o_{Base} = \phi^o \omega_V V_{Base}, \quad \omega_V = 1 + \frac{\mu}{\phi^o} C_{2,T} \quad C_{2,T} = 0.067 + 0.4(T_i - 0.5) \leq 1.15$$

# TORSIONALLY UNRESTRAINED AND RESTRAINED BUILDINGS




Torsionally unrestrained




Torsionally restrained

If wall 1 (or 2) yields first, the system has zero rotational stiffness, and swings about the other wall. (Except that mass torsional inertia restrains response)

Orthogonal walls provide rotational restraint

# PREDICTING TORSIONAL RESPONSE (2)

- Translational response of C.of M. from DDBD

- Rotation of floor plan: $$\theta_N = V_{Base} \cdot e_R / J_{R,\mu}$$

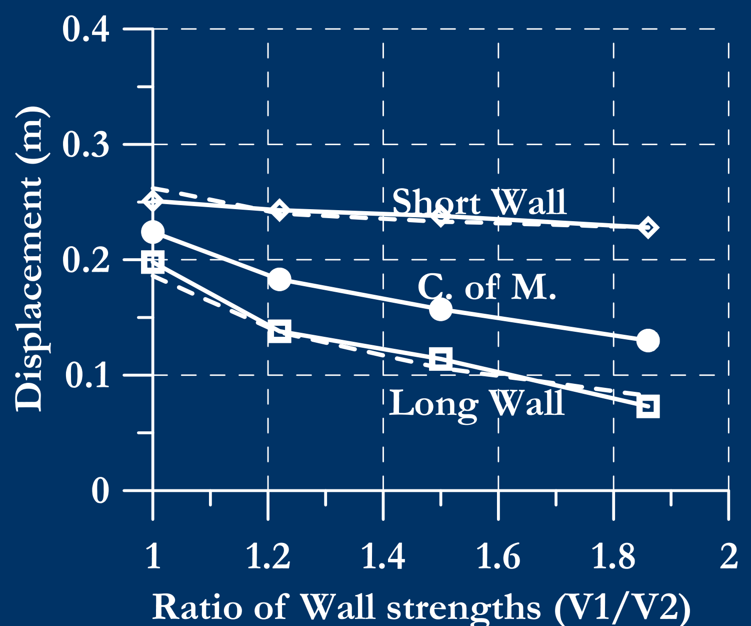
- Effective torsional stiffness: $$J_{R,\mu} = \sum_1^n \frac{k_{Zi}}{\mu_{sys}}(x_i - e_{RX})^2 + \sum_1^n k_{Xi}(z_i - e_{RZ})^2$$

- Displacements of end walls are given by: $$\Delta_i = \Delta_{CM} + \theta_N \cdot (x_i - e_V)$$

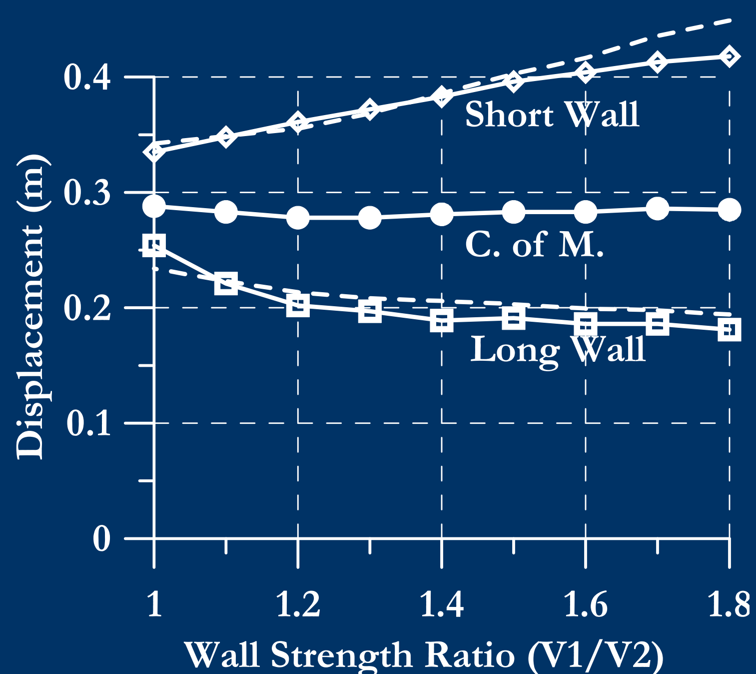
- (Note distance from C.of STRENGTH used)

# PREDICTION OF TORSIONAL RESPONSE (3)

Predictions dashed lines; time-history results solid lines + data points




Castillo's TU system, with system strength increase

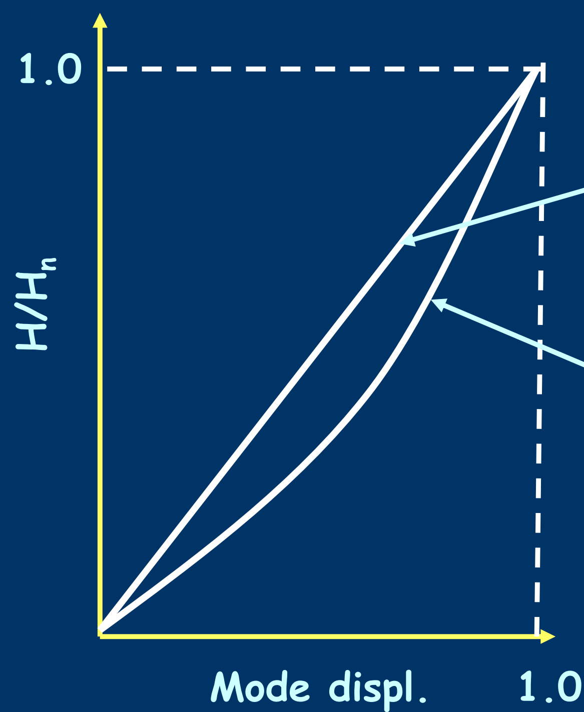



Beyer's TR system, with constant strength

# Moment Frames

- Displaced shapes for frames (target and yield)
- Damping for Moment Frames
- Higher Modes
- Irregular Frames and the displaced shape
- Structural Analysis
- Gravity and Seismic Moments

# CRITICAL DISPLACEMENTS: FRAMES




n≤4: $\delta_i = H_i / H_n$

n>4: $\delta_i = \frac{4}{3} \cdot \left( \frac{H_i}{H_n} \right) \cdot \left( 1 - \frac{H_i}{4H_n} \right)$

(Based on results of inelastic time-history analysis. Note, in some cases it may be necessary to review the displacement profile after initial analysis)

# EQUIVALENT VISCOUS DAMPING

For buildings, determine yield displacement at the effective height of:

$$H_e = \sum_{i=1}^{n}(m_i \Delta_i H_i) / \sum_{i=1}^{n}(m_i \Delta_i)$$

Frames: $$\Delta_y = \theta_y . H_e$$

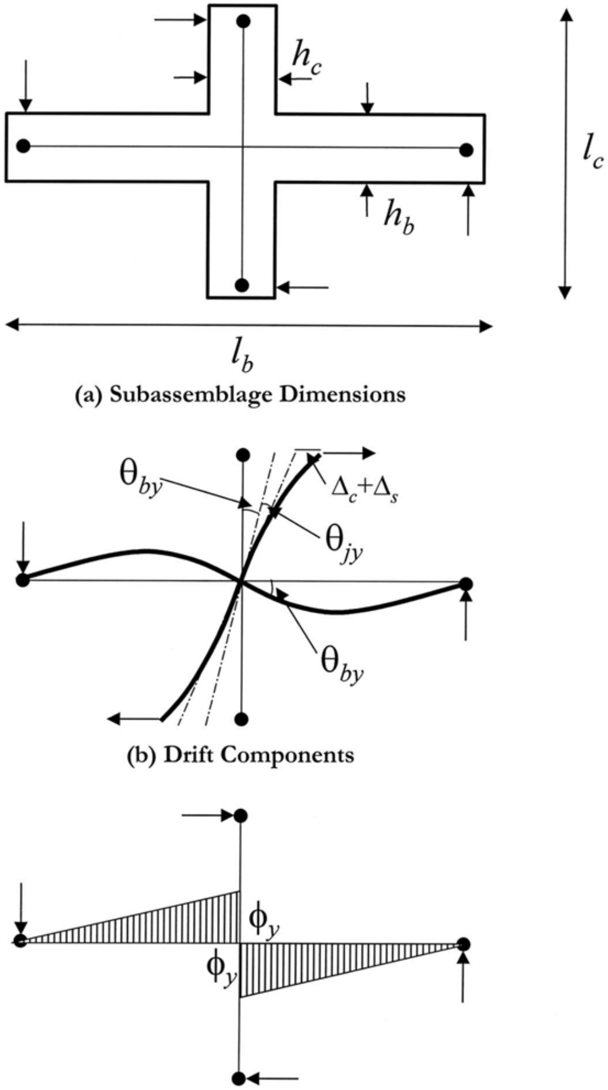
Walls: $$\Delta_{yi} = \frac{\varepsilon_y}{l_w} H_i^2 \left(1 - \frac{H_i}{3H_n}\right)$$ with $H_i = H_e$

Hence find displacement ductility; hence damping

# Equivalent Viscous Damping

For frame buildings, alternatively, determine the ductility of each storey, and hence damping of each storey.

Storey damping is then averaged to obtain total building damping (solution to one of sample problems done in this way. Textbook example utilizes previous approach)



(a) Subassemblage Dimensions

(b) Drift Components

(c) Beam Curvature Distribution

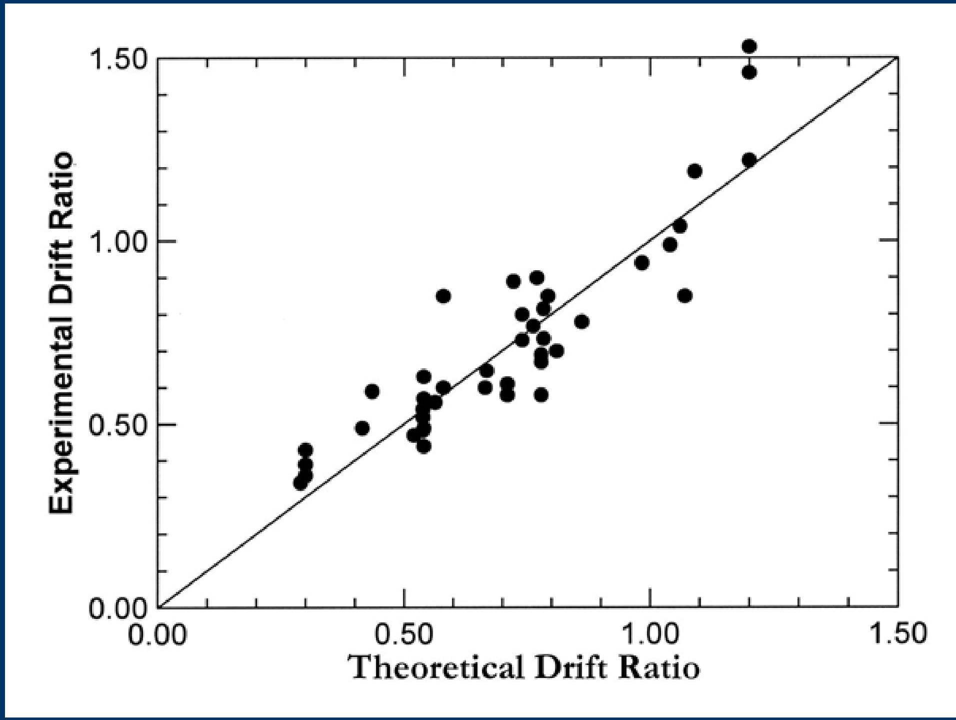
# YIELD DRIFTS OF FRAMES

## ELASTIC DEFORMATION CONTRIBUTIONS TO DRIFT OF A BEAM/COLUMN JOINT SUBASSEMBLAGE

Beam flexure and shear deform.

Column flexure and shear deform.

Joint shear deformation




$$\theta_y = 0.5\varepsilon_y(l_b/h_b)$$

**EXPERIMENTAL DRIFTS OF BEAM/COLUMN TEST UNITS COMPARED WITH EQUATION**

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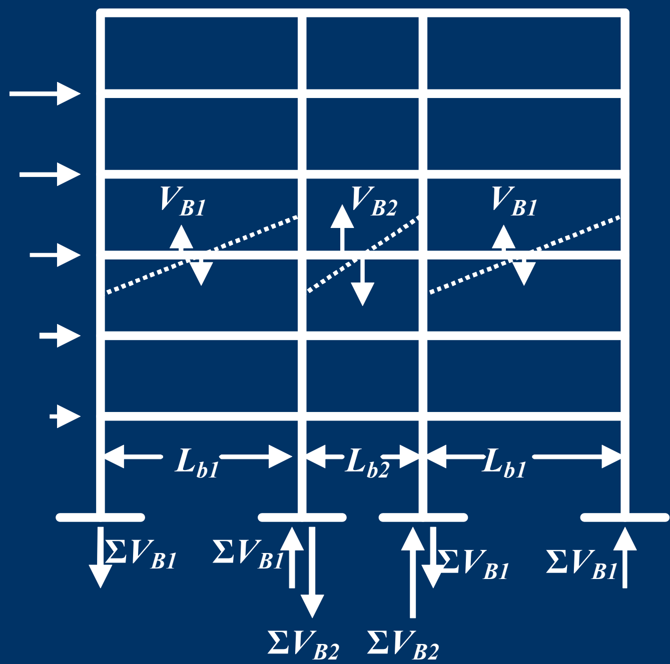
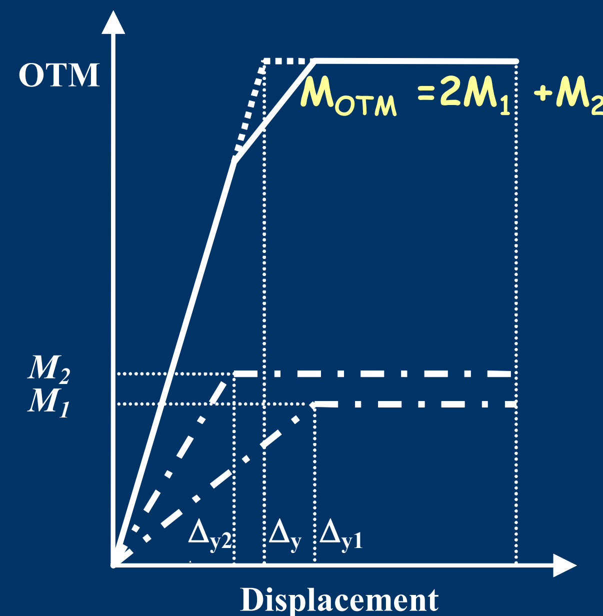
# YIELD DISPLACEMENT OF IRREGULAR FRAMES




(a) Irregular Frame



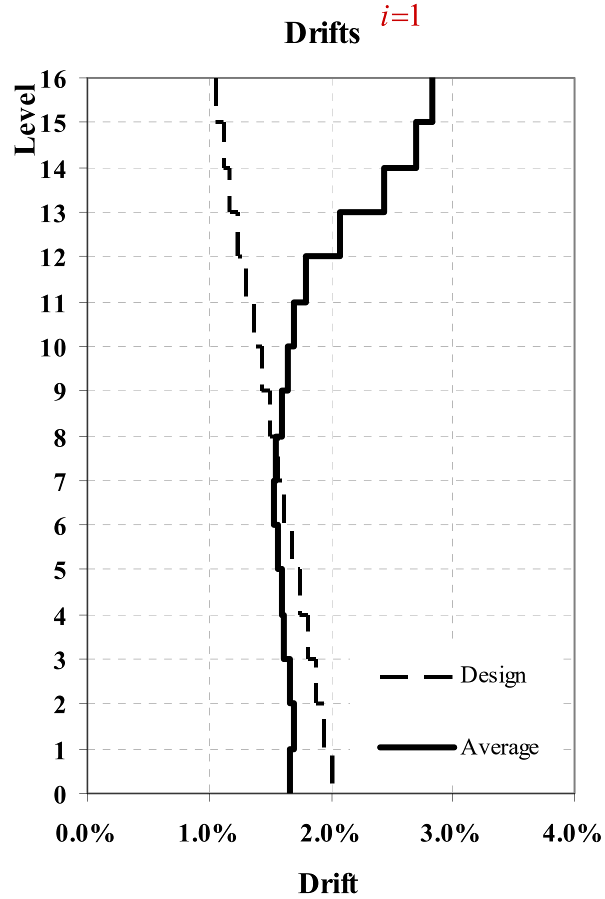

(b) Overturning Moment

$$\theta_{y1} = 0.5\varepsilon_y \frac{L_{b1}}{h_{b1}} \qquad \theta_{y2} = 0.5\varepsilon_y \frac{L_{b2}}{h_{b2}} \qquad \Delta_y = \frac{2M_1\theta_{y1} + M_2\theta_{y2}}{2M_1 + M_2} \cdot H_e$$

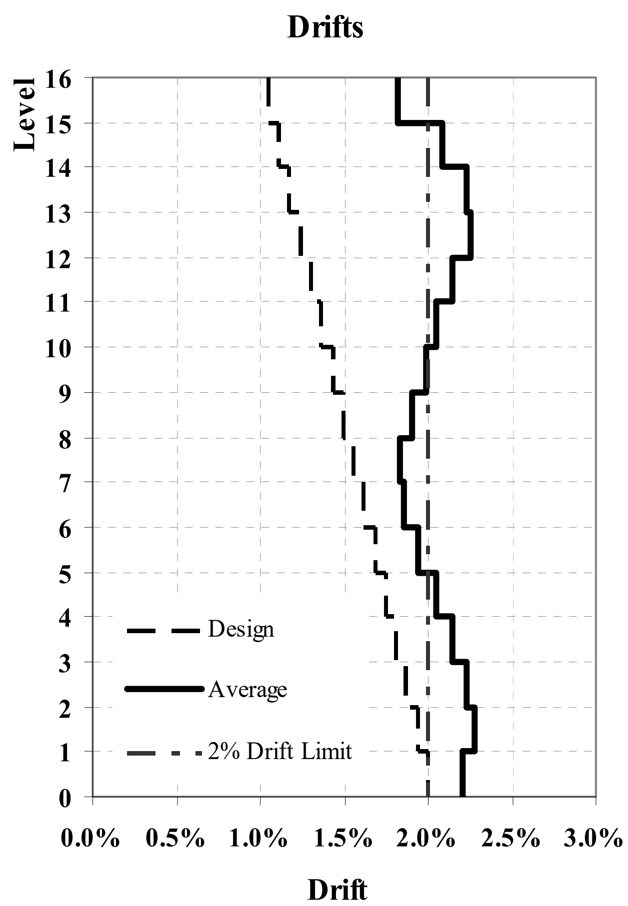
Note: If beam moment capacities in inner and outer frames equal, M1≈M2

# CONTROLLING HIGHER-MODE DRIFT AMPLIFICATION

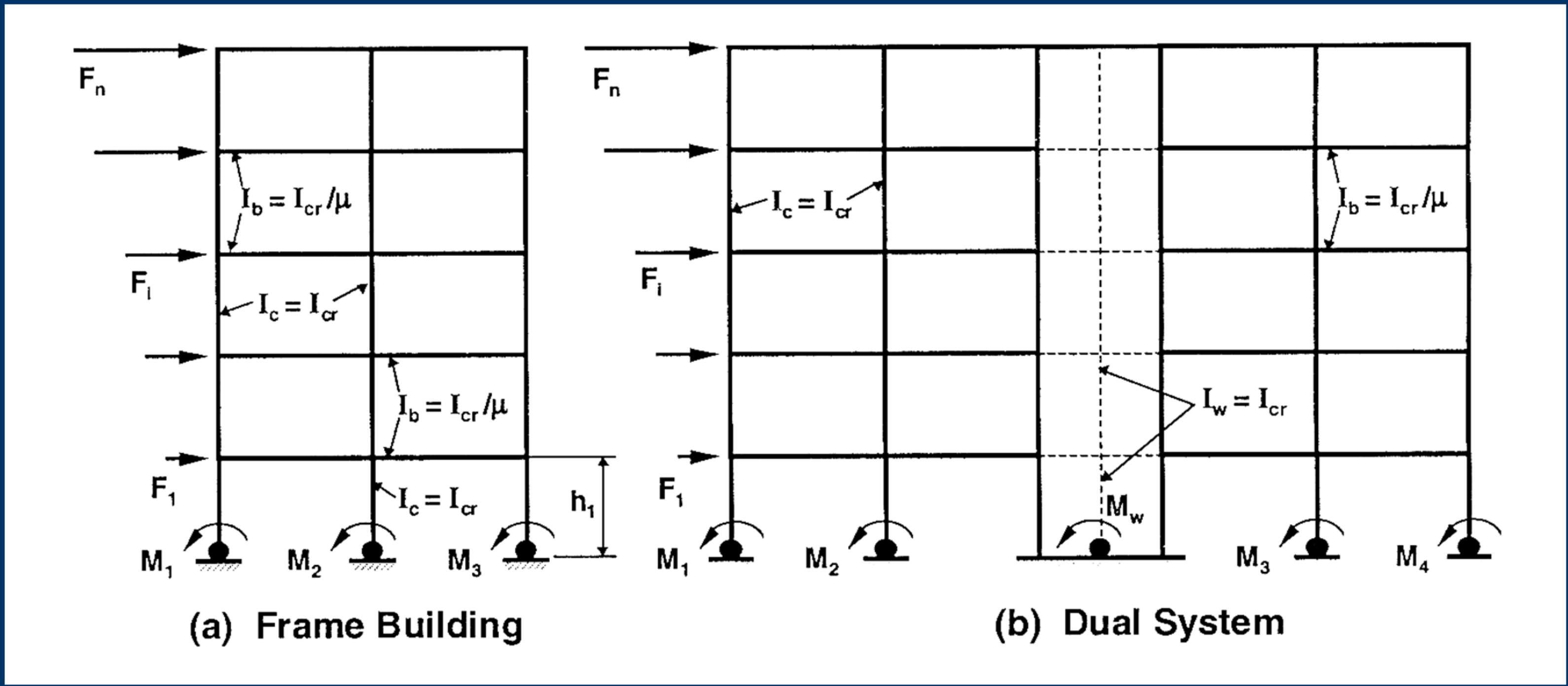
$$F_i = V_B (m_i \Delta_i) / \sum_{i=1}^{n} (m_i \Delta_i)$$




**0.1$V_{Base}$ at roof level**

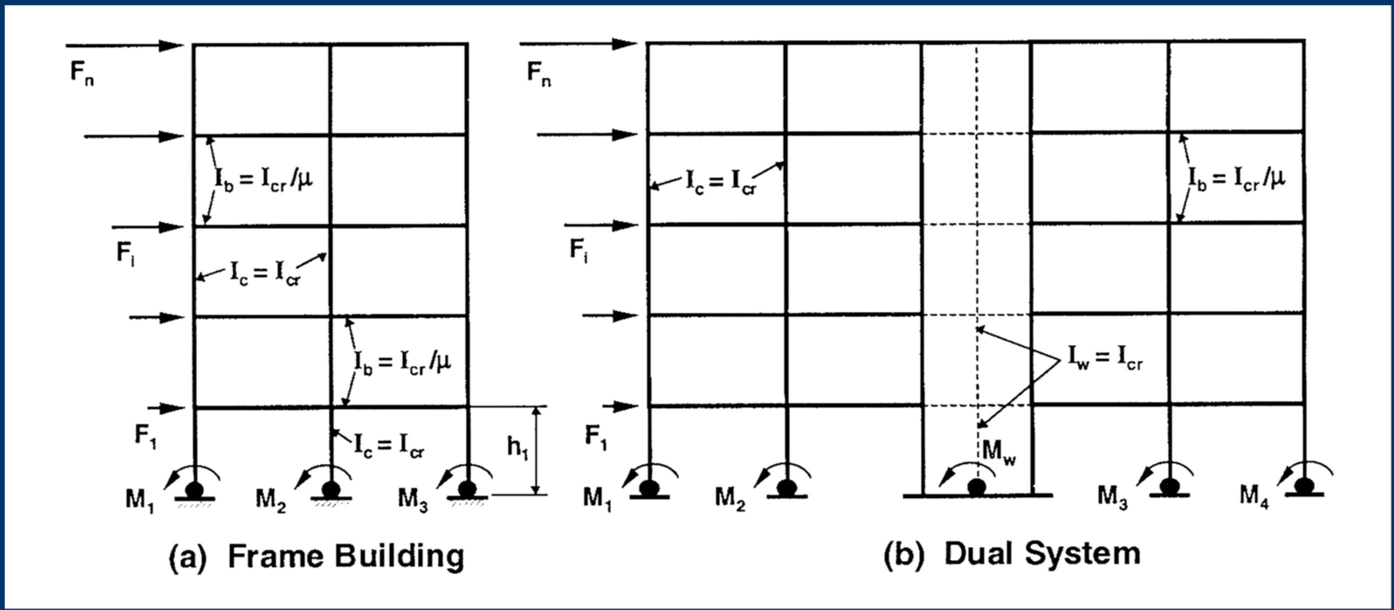


# ANALYSIS OF STRUCTURE UNDER DESIGN FORCES




**Beams:** Stiffness reduced by expected displacement ductility. **Columns:** Use cracked-section stiffness. **Walls:** Reduce stiffness in plastic hinge region by $\mu$.

# ANALYSIS OF STRUCTURE UNDER DESIGN FORCES -2




**Column Bases: Idealize as hinges, and chose base moment**

$$\sum M_b = M_1 + M_2 + M_3 = \sum_{i=1}^{n} F_i\left(0.6h_1\right) = V_b\left(0.6h_1\right)$$

## COMBINATION OF SEISMIC AND GRAVITY ACTIONS

Current practice is to add "reduced" gravity moments to seismic moments at plastic hinge locations. This results in unnecessary conservatism in design, meaning that the structure will respond at less than the intended limit-state deflection.

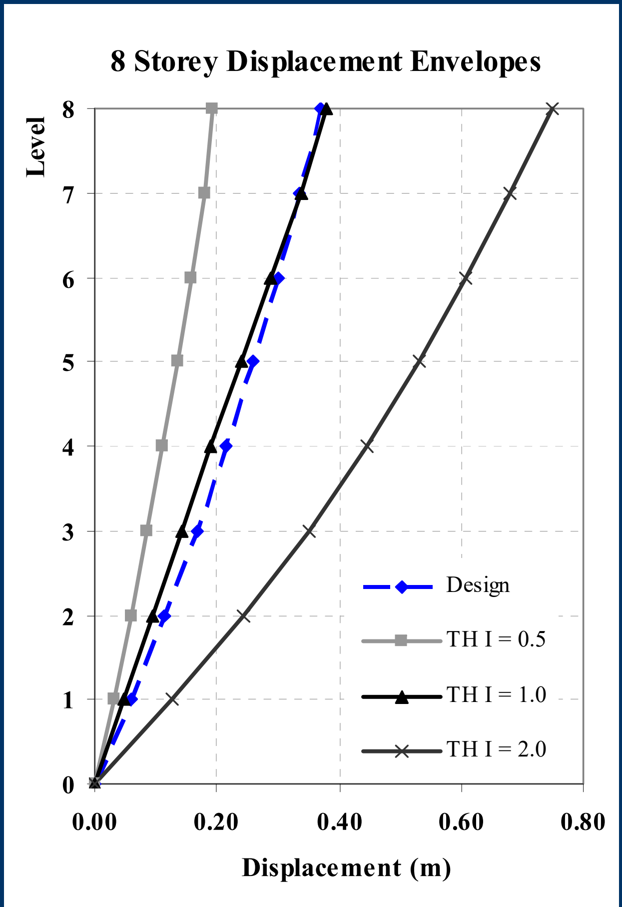
Although this is recognized to some extent in some codes by permitting limited moment-redistribution, it is not the usual case.

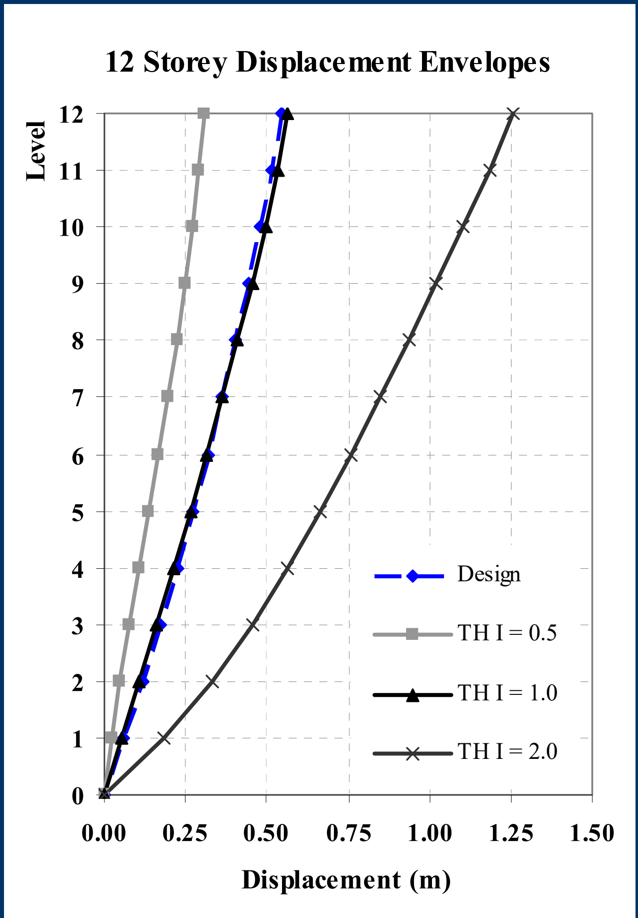
# BEAM DESIGN MOMENTS

Recommendation:

- Calculate factored gravity beam moments
- Calculate seismic beam moments
- Design for larger of gravity and seismic – do not add (codes require addition, but without any justification).

# DESIGN VERIFICATION –DISPLACEMENT PROFILES



# IMPLEMENTATION OF DDBD IN PRACTICE

**CODES:**

1. Chapter 14 is written in code format, as a starting point for code development.
2. Envisaged as being adopted as an "alternative" design process (i.e. parallel force-based and displacement-based procedures acceptable)
3. POLA has adopted DDBD as the preferred procedure of a dual force-based/displacement-based code (performance defined by limit strains for BOTH approaches.
4. Australia, New Zealand, Europe developing DDBD based codes.
5. Upcoming AASHTO PBSE Guidelines will include DDBD as an alternative.

# IMPLEMENTATION OF DDBD IN PRACTICE

WITHIN EXISTING CODES:

• Many codes permit the use of Time-history analysis as a design tool, hence:

1. Design using DDBD to obtain a rational design

2. Verify response using ITHA.

Note that this approach follows the logical procedure of using analysis to verify design, rather than using analysis to DEFINE design (as with multi-modal analysis)

# Path Forward…

The 'Diegos' have just completed their PhD degrees on damping models and bar buckling.

Julio is studying external pocket connections for ABC

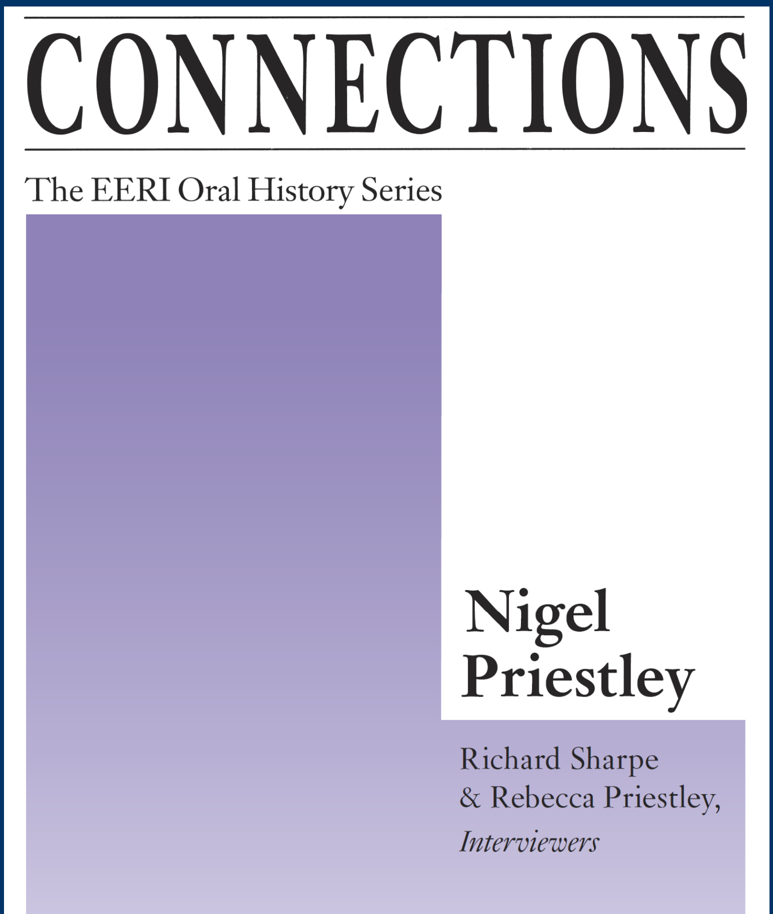
Ariadne's PhD is on bi-directional DDBD.

She has modernized our DDBD slide deck and has co-taught short courses on DDBD.

Jessi and Lina are studying high strength steel, including strain limit state definitions and low temperature impacts

Victor, Taylor, and Ana include DDBD within their work on condition, repair, and assessment

# Closing Argument..




***"If we can quantify the strain limits that define a performance level, then we could determine what the displacements would be, because one can integrate a strain field representing a building and determine what the displacements are. If we could do this, then we could relate strains to displacement to structural strength that would provide a structure which satisfied the design performance criteria. But it comes out that if you do this sort of approach, there are very few assumptions that you have to make in the design and you come up with something which is intellectually satisfying as well as gives some assurance of acceptable performance during the design-level earthquake. This you cannot get with current force-based seismic design."***

**Prof. Nigel Priestley speaking with daughter Prof. Rebecca Priestley in 2014 about DDBD**