CE 75.03 – Earthquake Engineering for Tall Buildings

In-class Session

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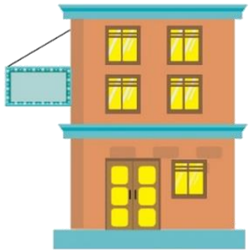
Topics for this Session

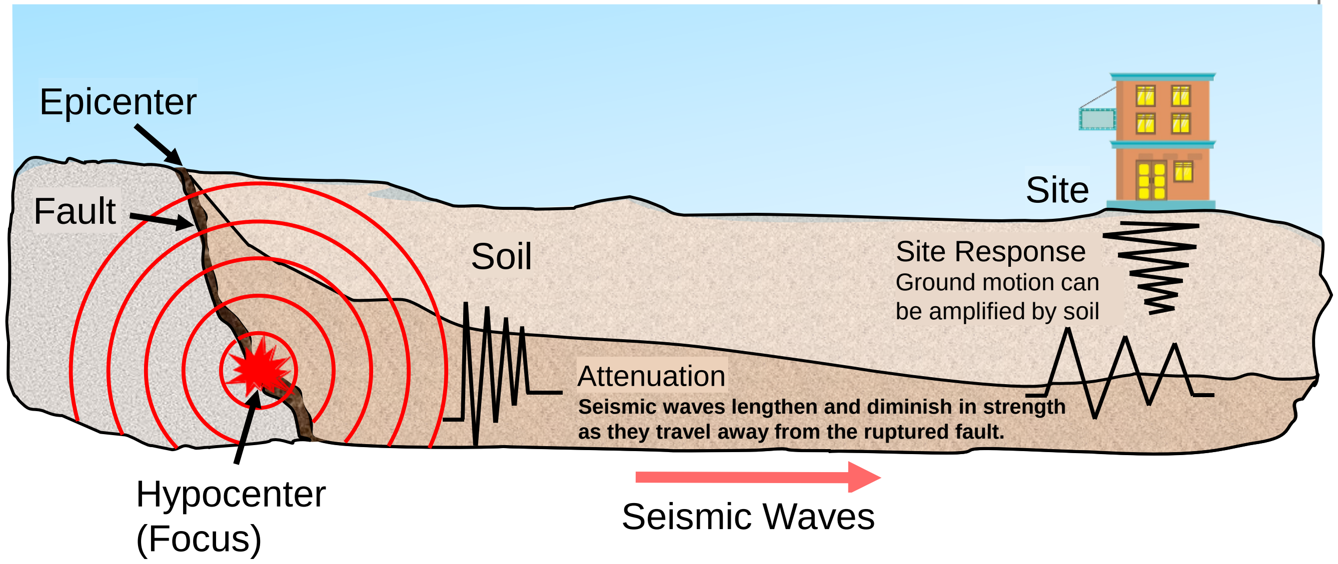
1) Introduction to the Nonlinear Static (Pushover) Analysis Procedures
2) Approximate Multi-mode based Seismic Analysis Procedures
    a) The Modal Pushover Analysis (MPA) Procedure
    b) The Uncoupled Modal Response History Analysis (UMRHA) Procedure
    c) The Modified Response Spectrum Analysis (MRSA) Procedure

The Earthquake Problem

Structure



Future ground shaking



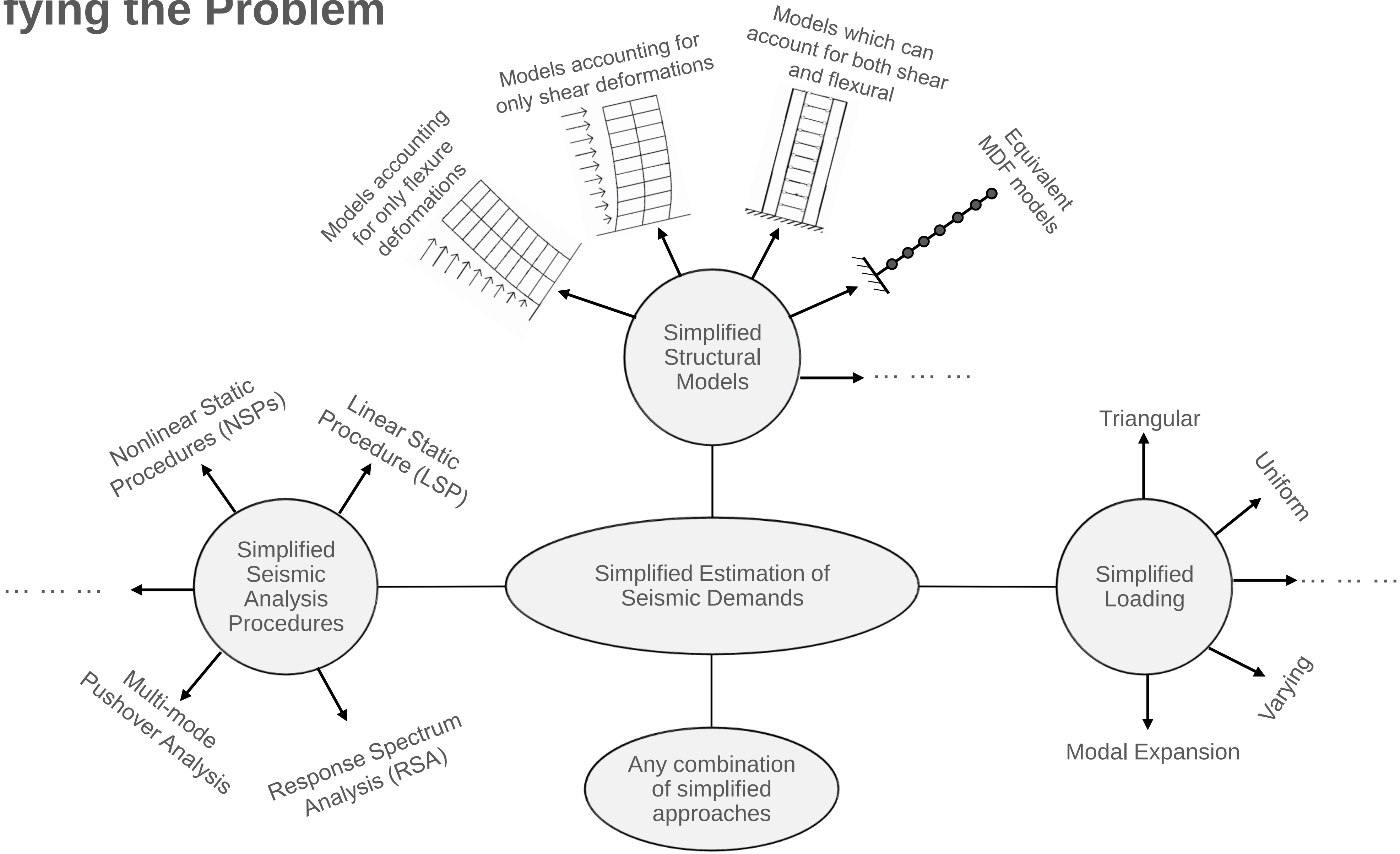
Linear/Nonlinear Analysis Model

Characterization of Seismic Ground Motions

Estimation of Linear/Nonlinear Seismic Demands

- Global-level Responses
- Inter-story Responses
- Component-level Responses

Simplifying the Problem

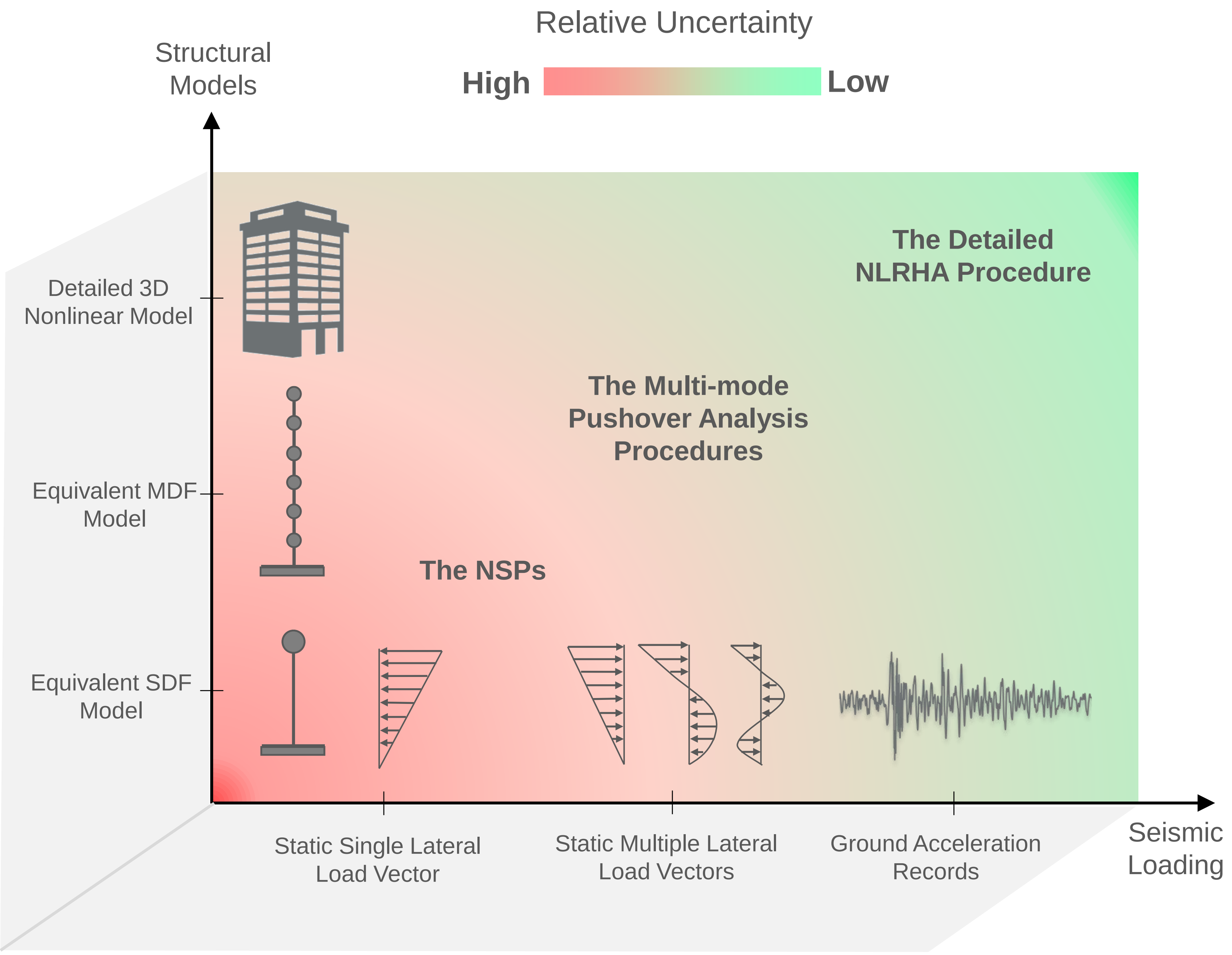


Seismic Analysis Procedures

Structural Model / Seismic Loading	**Linear** E, A, I, L, G etc. = Constant, K= Constant	**Nonlinear** E ≠ Constant, EI ≠ Constant, K≠ Constant
Static	**1.** Equivalent Lateral Force (ELF) Procedure	**5.** Several Pushover Analysis Methods or Nonlinear Static Procedures (NSPs)
Dynamic	**2.** Response Spectrum Analysis (RSA) Procedure (or Mode Spectral Analysis) **3.** Modal Response History (or Time History) Analysis Procedure (Modal RHA/THA) **4.** Linear Response History (or Time History) Analysis Procedure (Direct Integration Linear RHA /LTHA)	**6.** Nonlinear Modal Response History Analysis or Fast Nonlinear Analysis (FNA) **7.** Nonlinear Response History (or Time History) Analysis Procedure (Direct Integration Nonlinear RHA/THA)

Determination of Nonlinear Seismic Demands

Analysis Procedures



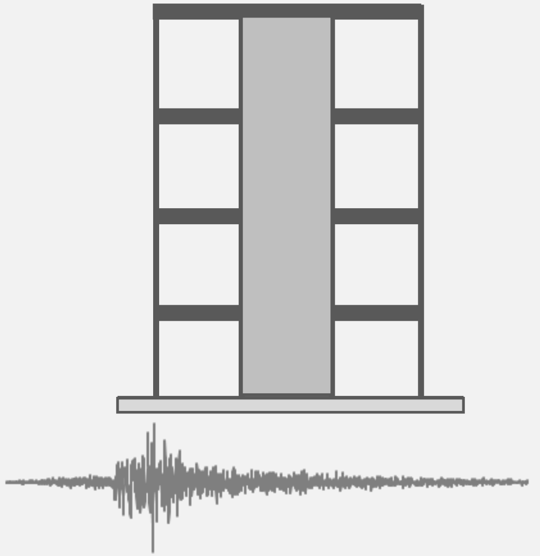
Introduction to the Nonlinear Static (Pushover) Analysis Procedures

Nonlinear Static Analysis Procedures (NSPs) – Pushover Analysis Procedures

- The nonlinear analysis recommended for the first generation of performance-based seismic design methodology. Currently, it can be regarded as an alternate method of analysis for carrying out the PBD.

Basic Idea of Pushover Analysis

Full 3D Nonlinear MDF Model



A ground motion structure in a particular X or Y direction

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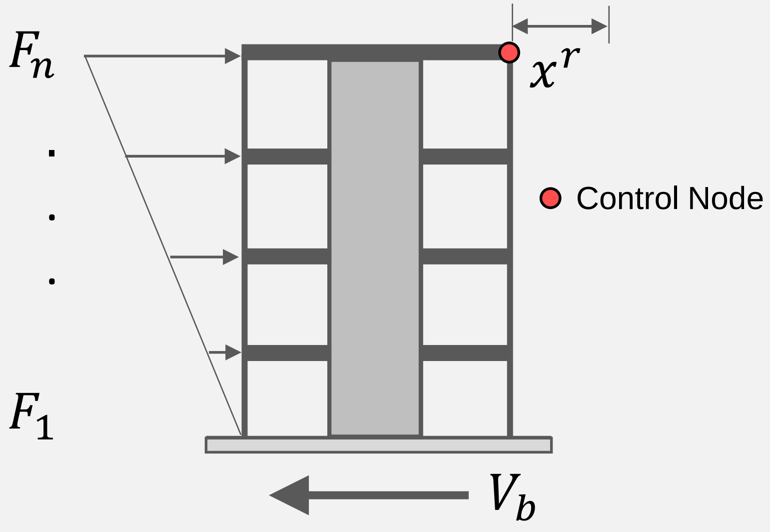
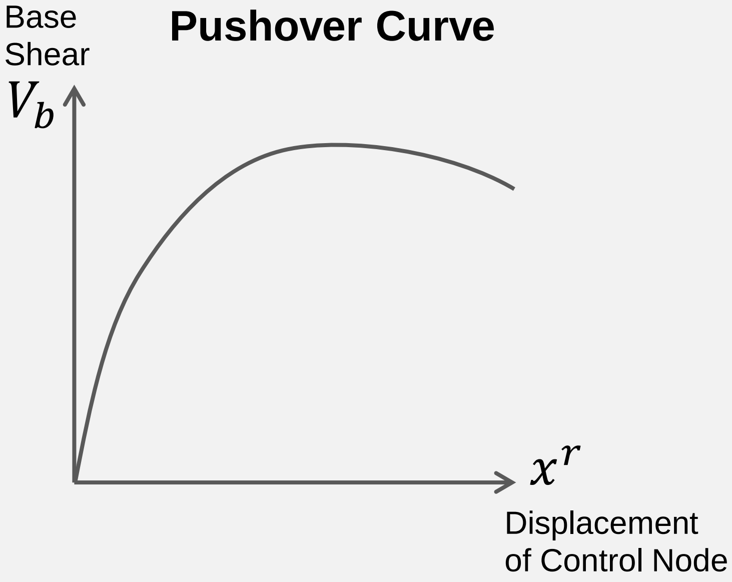
Monotonic Pushover Analysis



Monotonically increasing lateral load pattern F_1 … … F_n to push the structure in that particular X or Y direction

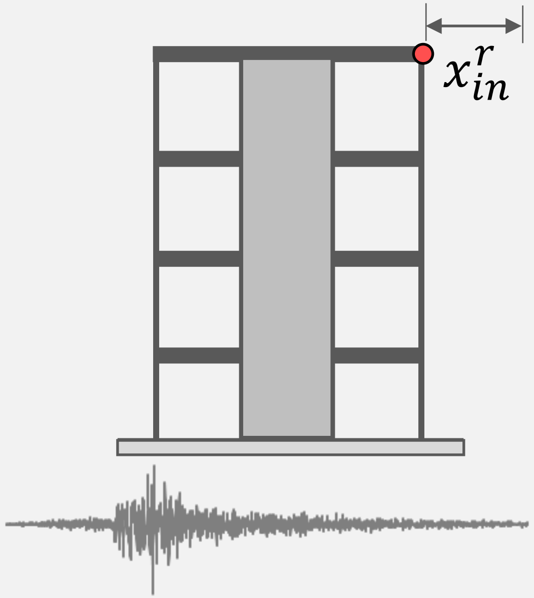
Pushover Curve



Develop the V_b vs. x^r relationship

Nonlinear Static Analysis Procedures (NSPs) – Pushover Analysis Procedures

Full 3D Nonlinear MDF Model



A ground motion in a particular X or Y direction

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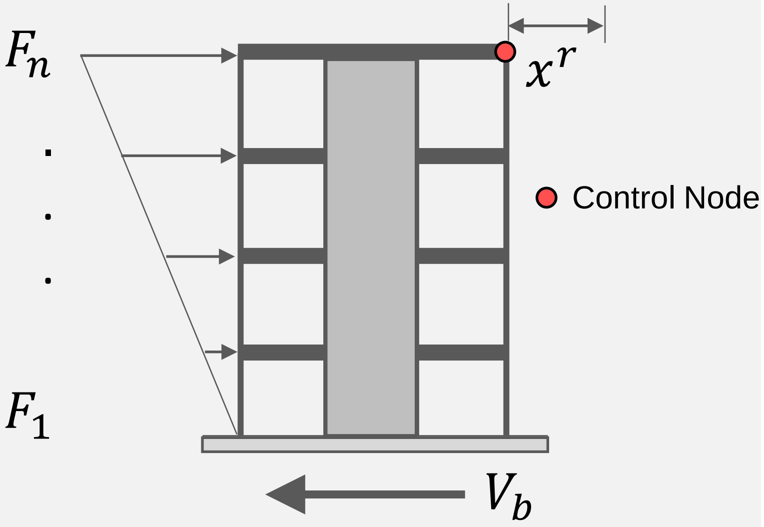
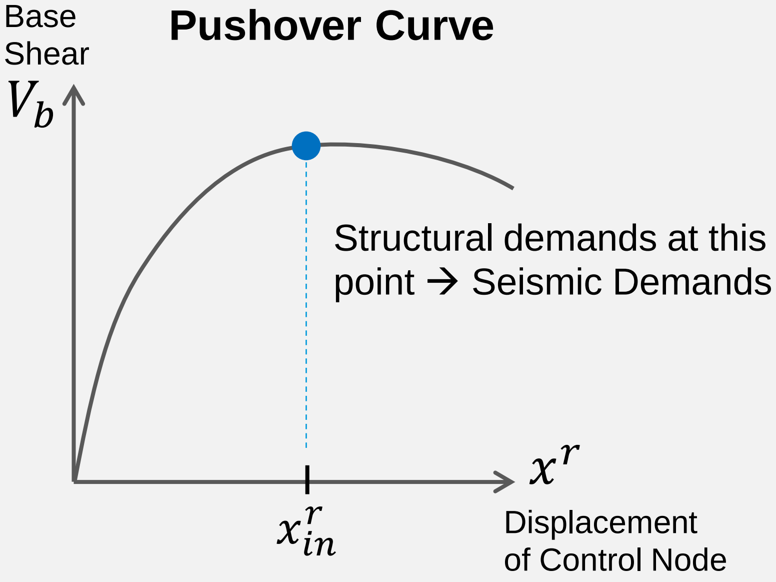
Monotonic Pushover Analysis



Monotonically increasing lateral load pattern F_1 … … F_n to push the structure in that particular X or Y direction

Pushover Curve



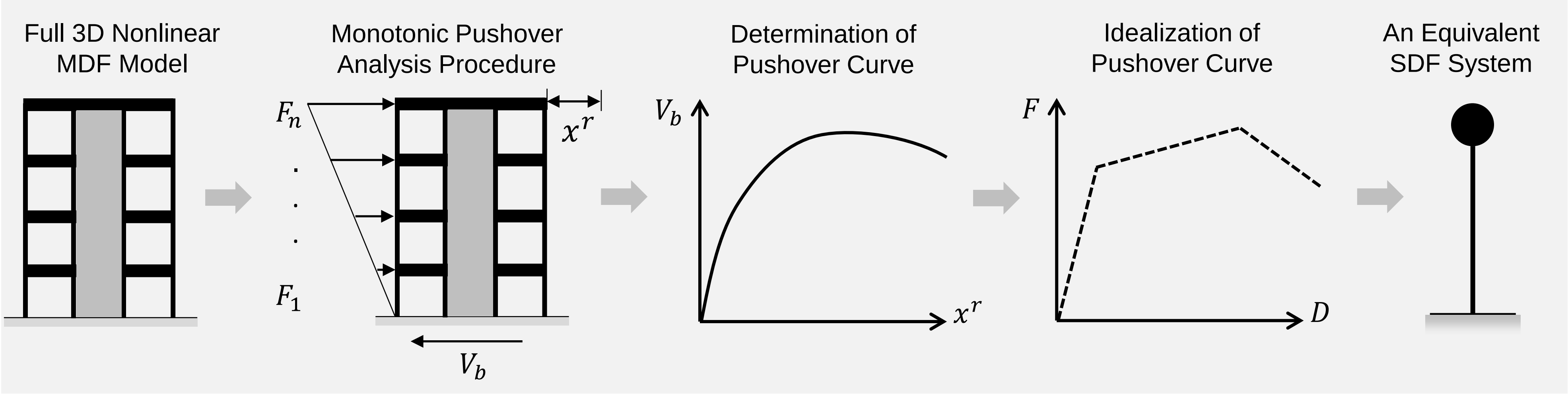
Develop the V_b vs. x^r relationship

Let's set the control node as a roof node and denote the peak inelastic roof displacement occurred during the ground motion as x^r_{in}.

During the push, when $x^r = x^r_{in}$, the force and displacement demands of the structure are the peak seismic demands (produced by the ground motion).

How to Determine x^r_{in}?

Nonlinear Static Procedures



The Concept of an “Equivalent SDF System”

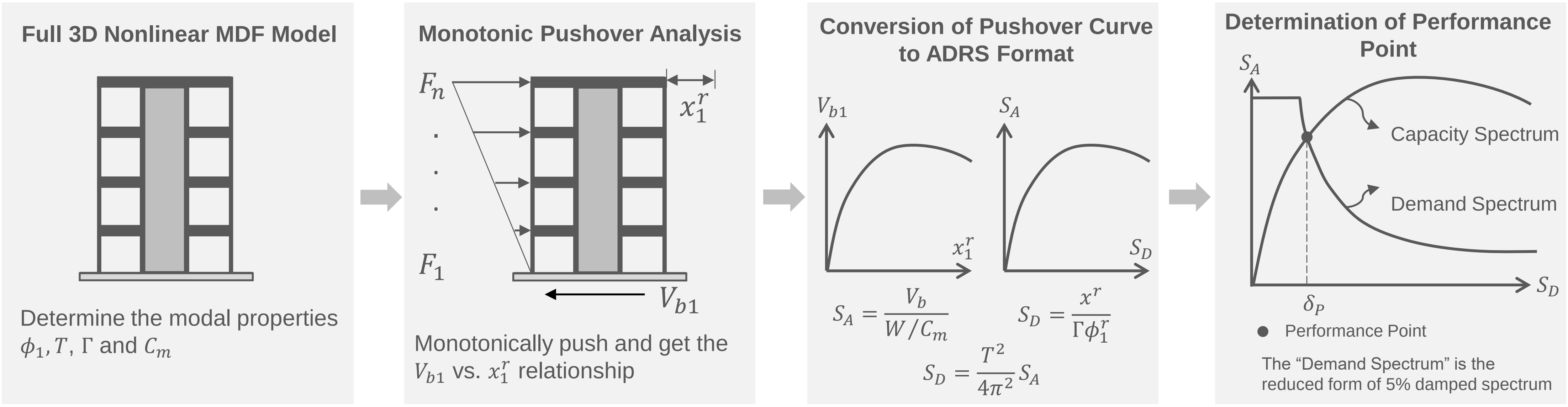
Equivalent Linearization

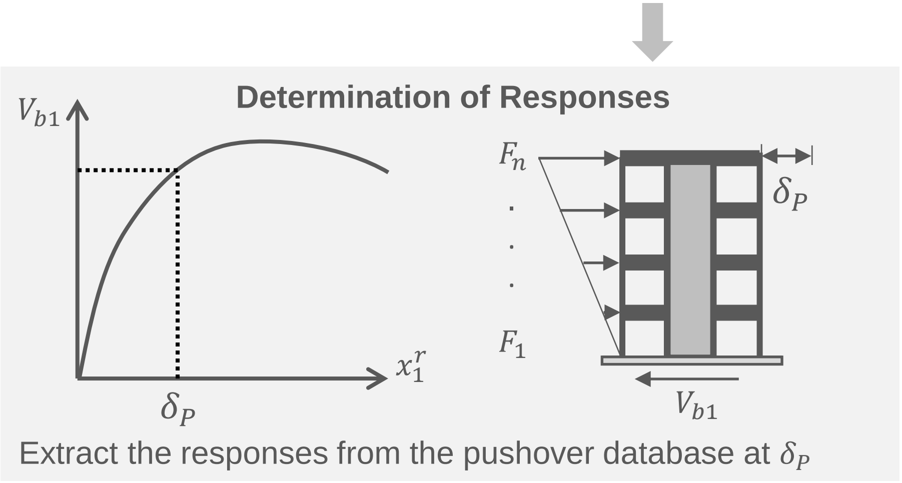
- Several EL Procedures (Individual studies)
- Capacity Spectrum Method (CSM) (ATC 40, FEMA 273, FEMA 356)
- FEMA 440 Improved EL Procedure

Displacement Modification

- Several expressions for displacement modification (Individual studies)
- Displacement Coefficient Method (ATC 40, FEMA 273, FEMA 356, FEMA 440, ASCE 41-06, ASCE 41-13)

Capacity Spectrum Method (ATC 40, 1996)

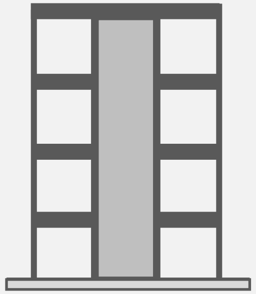




- **Capacity Spectrum** → Another form of pushover curve (SA-SD form).
- **Demand Spectrum** → Another form of response spectrum (ADRS Form) reduced based on effective damping (i.e. original inherent damping + additional hysteretic damping)

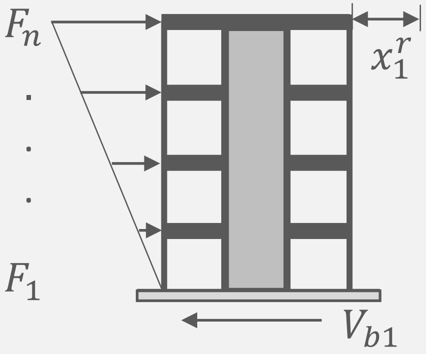
FEMA 440 (2005) Displacement Coefficient Method

Full 3D Nonlinear MDF Model



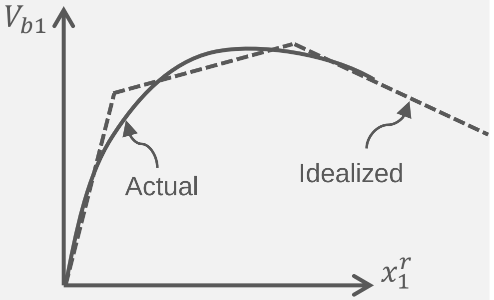
- Determine the modal properties ϕ_1, T, Γ and C_m

Monotonic Pushover Analysis



- Monotonically push and get the V_{b1} vs. x_1^r relationship

Idealization of Pushover Curve



- Idealized force-displacement relationship.
- Determine K_e, V_y and R_y

Determination of Target Displacement

- Determine the effective time period and coefficients

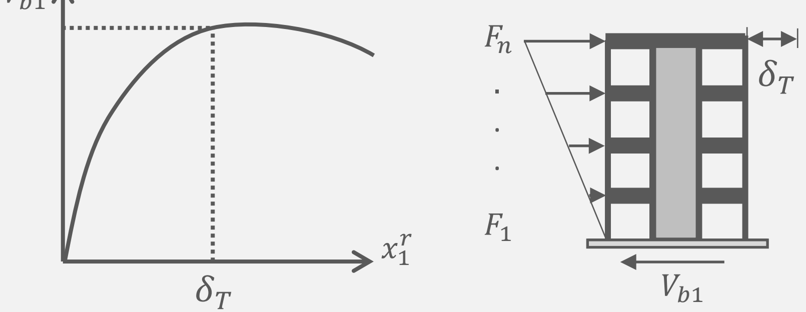
$$T_e = T_i\sqrt{\frac{K_i}{K_e}} \qquad C_1 = 1 + \frac{R_y - 1}{aT_e^2}$$

$$C_2 = 1 + \frac{1}{800}\left(\frac{R_y - 1}{T_e}\right)^2$$

- Target displacement

$$\delta_T = C_o C_1 C_2 S_A \frac{T_e^2}{4\pi^2} g$$

Determination of Responses



- Extract the responses from the pushover database at δ_T

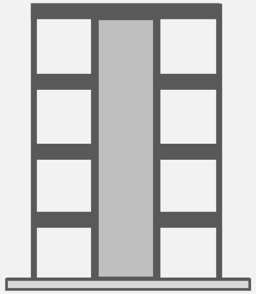
Target displacement:

$$\delta_T = C_o C_1 C_2 S_A \frac{T_e^2}{4\pi^2} g$$

$$T_e = T_i\sqrt{\frac{K_i}{K_e}}, \qquad C_1 = 1 + \frac{R_y - 1}{aT_e^2}, \qquad C_2 = 1 + \frac{1}{800}\left(\frac{R_y - 1}{T_e}\right)^2$$

ASCE/SEI 41-17 (2017) Nonlinear Static Procedure

Full 3D Nonlinear MDF Model



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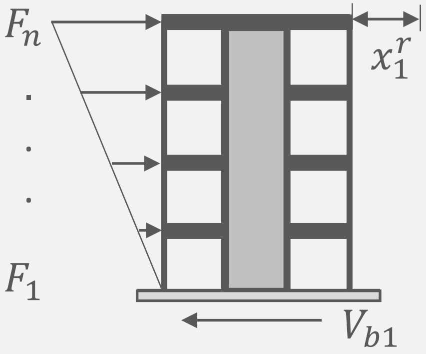
Monotonic Pushover Analysis



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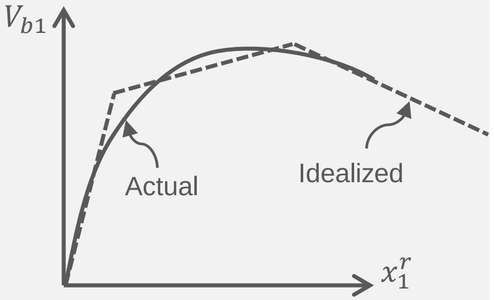
Idealization of Pushover Curve



- Idealized force-displacement relationship.
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Determination of Target Displacement

- Determine the effective time period and coefficients

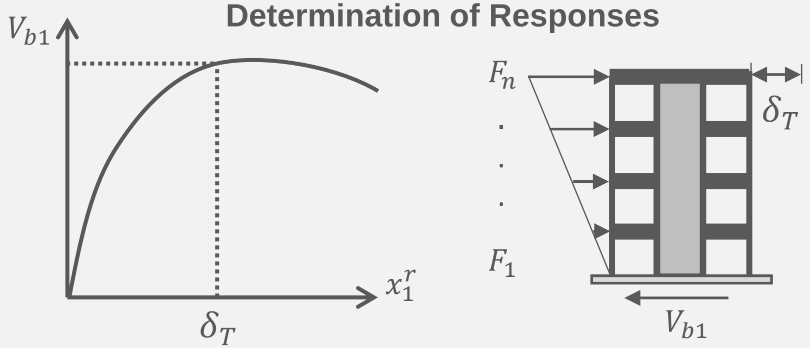
$$T_e = T_i\sqrt{\frac{K_i}{K_e}} \qquad C_1 = 1 + \frac{\mu_{strength} - 1}{aT_e^2}$$

$$C_2 = 1 + \frac{1}{800}\left(\frac{\mu_{strength} - 1}{T_e}\right)^2$$

- Target displacement

$$\delta_T = C_o C_1 C_2 S_A \frac{T_e^2}{4\pi^2} g$$

Determination of Responses



- Extract the responses from the pushover database at δ_T

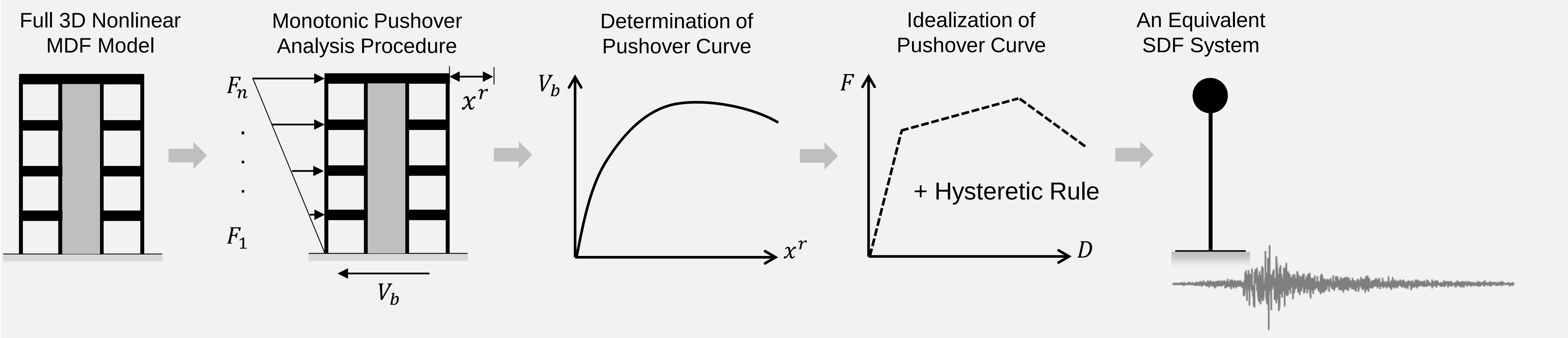
Target displacement:

$$\delta_T = C_o C_1 C_2 S_A \frac{T_e^2}{4\pi^2} g$$

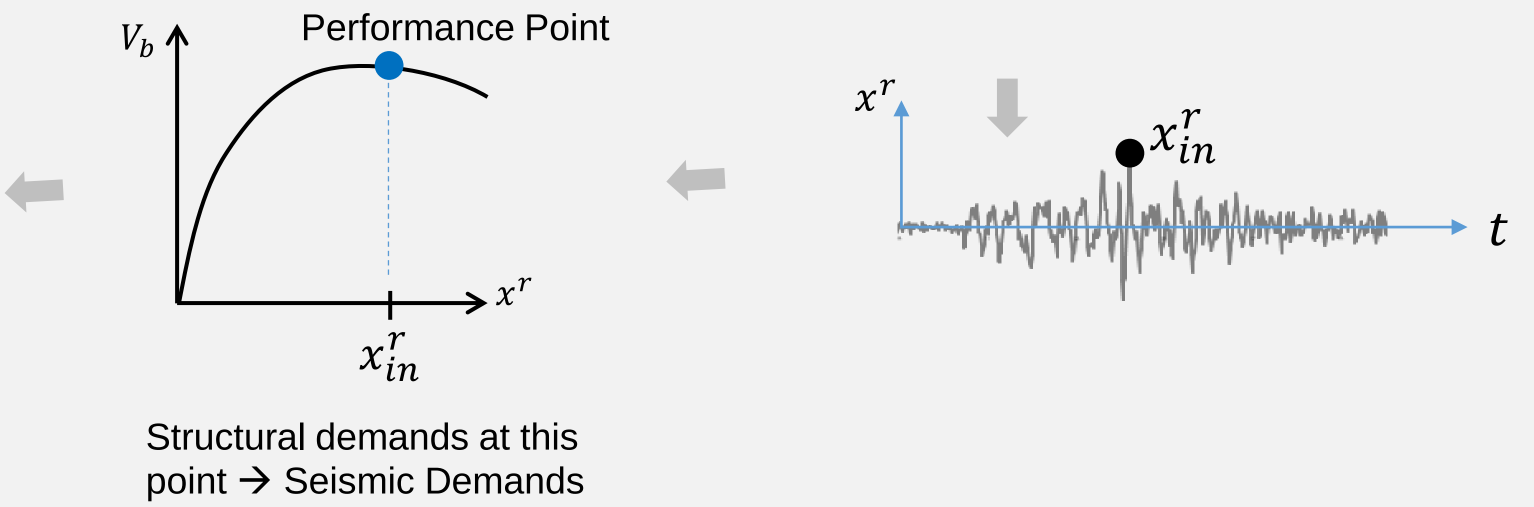
$$T_e = T_i\sqrt{\frac{K_i}{K_e}}, \qquad C_1 = 1 + \frac{\mu_{strength} - 1}{aT_e^2}$$

$$C_2 = 1 + \frac{1}{800}\left(\frac{\mu_{strength} - 1}{T_e}\right)^2$$

Determining the Performance Point using NLRHA of the Equivalent SDF System

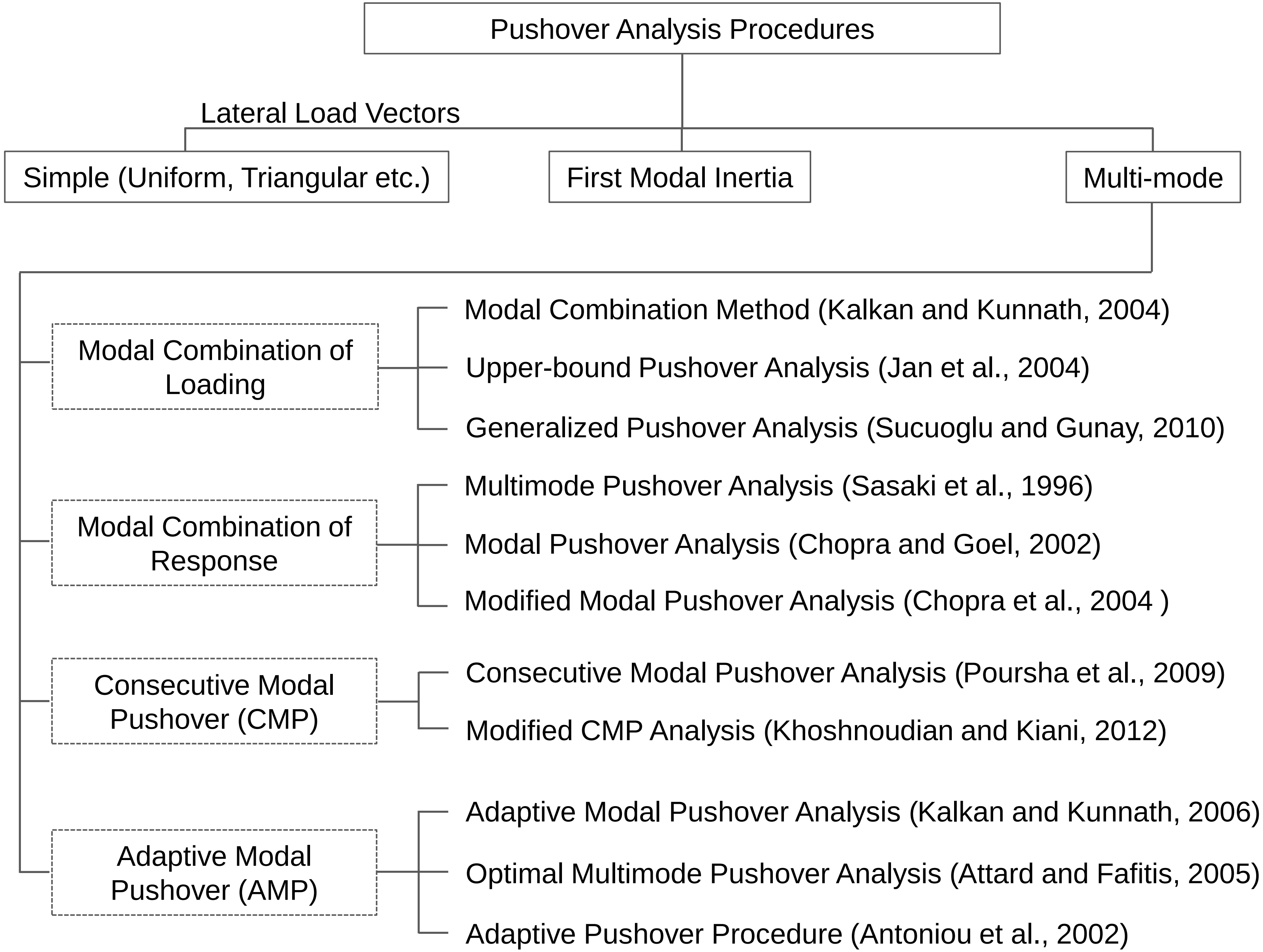


Extract all results (structural demands) from the software at that particular step of pushover analysis (i.e. corresponding to the performance point or target displacement).



Approximate Multi-mode based Seismic Analysis Procedures

Multi-mode Pushover Analysis Procedures



Why We Still Need Approximate Seismic Analysis Procedures?

Despite the development of fast computing tools, software and other advancements, the detailed nonlinear RHA is still a difficult task for several reasons.

- Ground motions compatible with the seismic design spectrum for the site must be selected.
- Computationally demanding, inelastic modeling, 3D analysis to account for coupling between lateral and torsional motions, subjected to 2 horizontal components of ground motions.
- Must be repeated for several excitations.
- Structural model must be sophisticated enough to represent a building realistically, especially deterioration in its strength at large displacements.

So, approximate methods are still an attractive option as an alternate to the rigorous NLRHA procedure.

Effective Earthquake Forces on an MDF System

The governing equation of motion for an elastic MDF system subjected to the earthquake ground motion is

$$\boldsymbol{M}\ddot{\boldsymbol{u}}(t) + \boldsymbol{C}\dot{\boldsymbol{u}}(t) + \boldsymbol{K}\boldsymbol{u}(t) = -\boldsymbol{M}\,\mathbf{1}\,\ddot{\boldsymbol{u}}_{\boldsymbol{g}}(\boldsymbol{t}) = \boldsymbol{P}_{\boldsymbol{eff}}(\boldsymbol{t})$$

Where the effective earthquake forces are

$$\boldsymbol{P}_{eff}(t) = -\boldsymbol{M}\,\mathbf{1}\,\ddot{u}_g(t)$$

The spatial distribution of these forces over the structure is defined by the vector

$$\boldsymbol{s} = \boldsymbol{M}\,\mathbf{1}$$

The idea of modal expansion of excitation vector $\boldsymbol{P}(t)$ of the form $\boldsymbol{P}(t) = \boldsymbol{s}\, p(t)$

$$\boldsymbol{P}(t) = \boldsymbol{s}\, p\,(t)$$

The primary idea is to expand the vector $\boldsymbol{s}$ as

$$\boldsymbol{s} = \sum_{r=1}^{N} \boldsymbol{s}_r = \sum_{r=1}^{N} \Gamma_r\, \boldsymbol{M}\, \phi_r$$

This equation may be viewed as an expansion of the distribution $\boldsymbol{s}$ of applied forces in terms of inertia force distributions $\boldsymbol{s}_r$ associated with natural modes.

Pre-multiplying both sides of above equation by $\boldsymbol{\phi}_n^T$ and utilizing the orthogonality property of modes gives

$$\Gamma_n = \frac{\boldsymbol{\phi}_n^T \boldsymbol{s}}{M_n}$$

The contribution of the nth mode to $\boldsymbol{s}$ is

$$\boldsymbol{s}_n = \Gamma_n \boldsymbol{M} \phi_n$$

Modal Expansion of the Effective Earthquake Forces

$$\boldsymbol{P}_{eff}(t) = -\boldsymbol{M}\,\boldsymbol{1}\,\ddot{u}_g(t) = -\boldsymbol{s}\,\ddot{u}_g(t)$$

- This force distribution can be expanded as a summation of modal inertia force distributions

$$\boldsymbol{s} = \boldsymbol{M}\,\boldsymbol{1} = \sum_{n=1}^{N} \boldsymbol{s}_n = \sum_{n=1}^{N} \Gamma_n \boldsymbol{M}\,\boldsymbol{\phi}_n$$

Where $\Gamma_n = \dfrac{L_n}{M_n}$

$$L_n = \phi_n^T \boldsymbol{M}\,\boldsymbol{1}$$

$$M_n = \phi_n^T \boldsymbol{M}\,\phi_n$$

Modal Expansion of the Effective Earthquake Forces

- The effective earthquake forces can then be expressed as

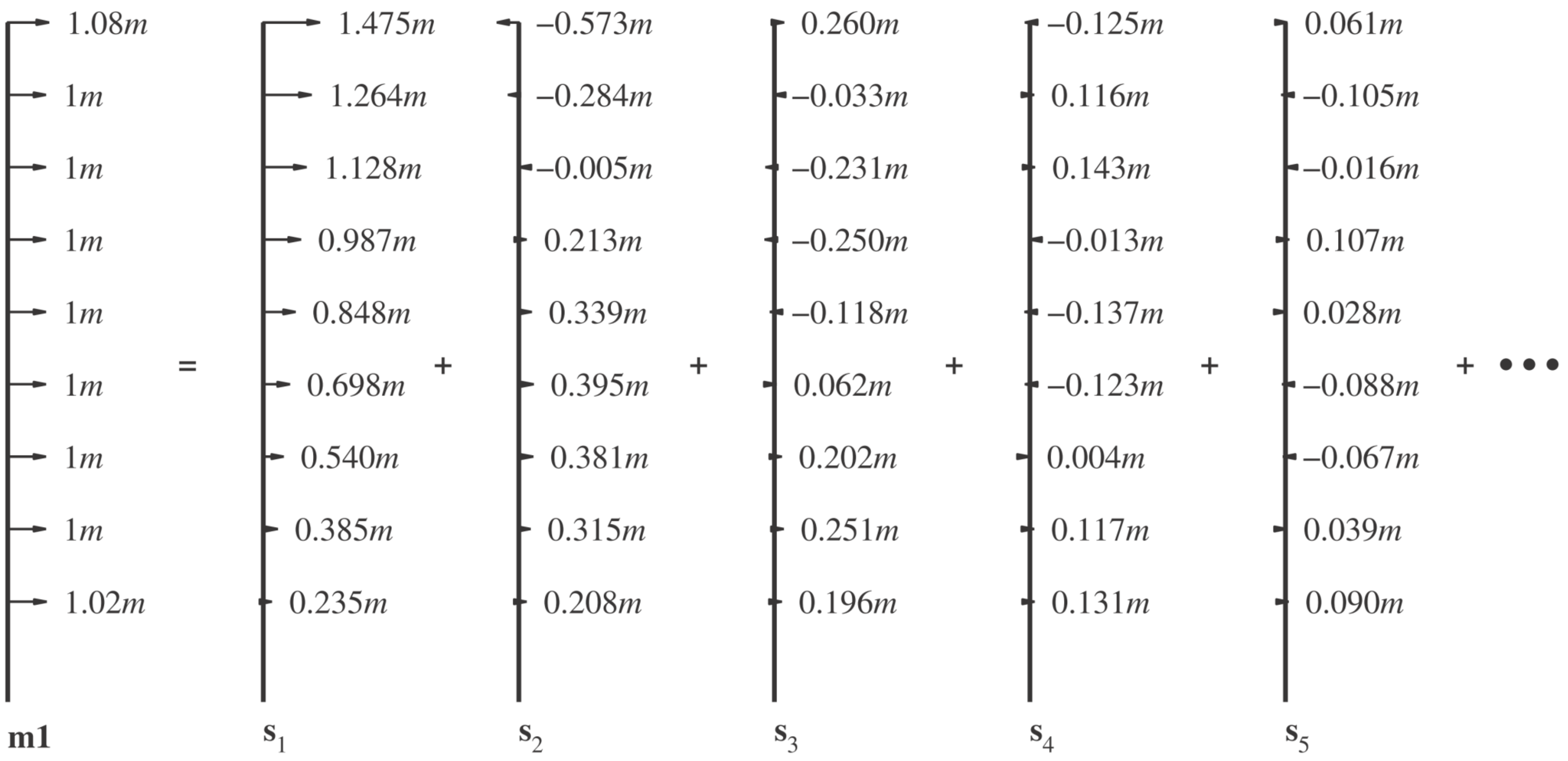
$$\boldsymbol{P}_{eff}(t) = \sum_{n=1}^{N} \boldsymbol{P}_{eff,n}(t) = \sum_{n=1}^{N} -\boldsymbol{s}_n \ddot{u}_g(t)$$

- The contributions of the n^{th} mode to $\boldsymbol{P}_{eff}(t)$ and $\boldsymbol{s}$ are

$$\boldsymbol{P}_{eff,n}(t) = -\boldsymbol{s}_n \ddot{u}_g(t)$$

$$\boldsymbol{s}_n = \Gamma_n \boldsymbol{M} \boldsymbol{\phi}_n$$

Modal expansion of the distribution $\boldsymbol{s} = \boldsymbol{M}\,\mathbf{1}$ of effective earthquake forces



The direction of force s_{jn} at the jth floor level is controlled by the algebraic sign of ϕ_{jn}, the jth-floor displacement in mode ϕ_n

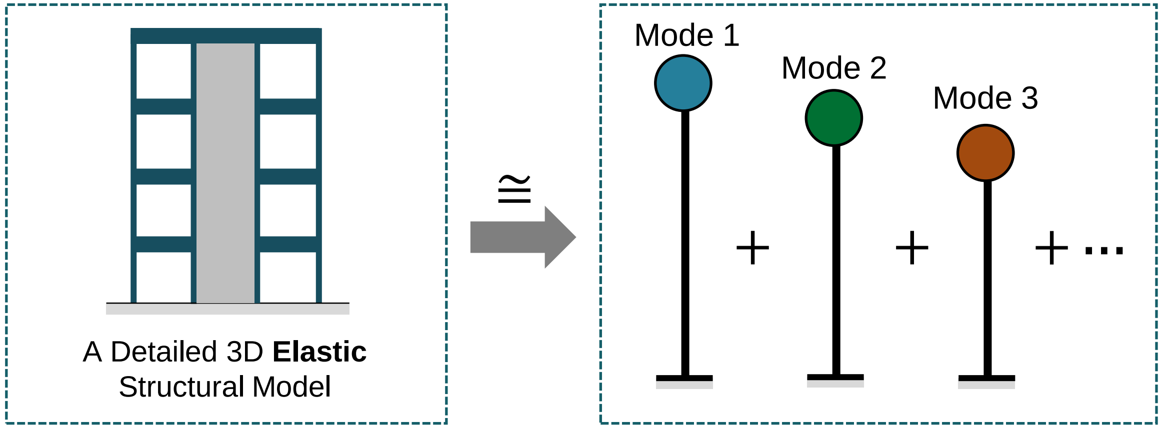
Modal Expansion of the Effective Earthquake Forces

- Utilizing the modal expansion of $\boldsymbol{P}_{eff}(t)$ and $\boldsymbol{s}$, two procedures for approximate analysis of inelastic buildings are proposed by Chopra and Goel (2002).
  - The uncoupled modal response history analysis (UMRHA), and
  - The modal pushover analysis (MPA)
- Not intended for practical application, the UMRHA procedure is developed only to provide a rationale for the MPA procedure.
- In the UMRHA procedure, the response history of the building to $\boldsymbol{P}_{eff,n}(t)$, the nth-mode component of the excitation, is determined by nonlinear RHA of an inelastic SDF system, and superposition of these “modal” responses gives the total response.
- In the MPA procedure, the peak response to $\boldsymbol{P}_{eff,n}(t)$ is determined by a nonlinear static, or pushover, analysis, and the peak modal responses are combined by modal combination rules to estimate the total response.

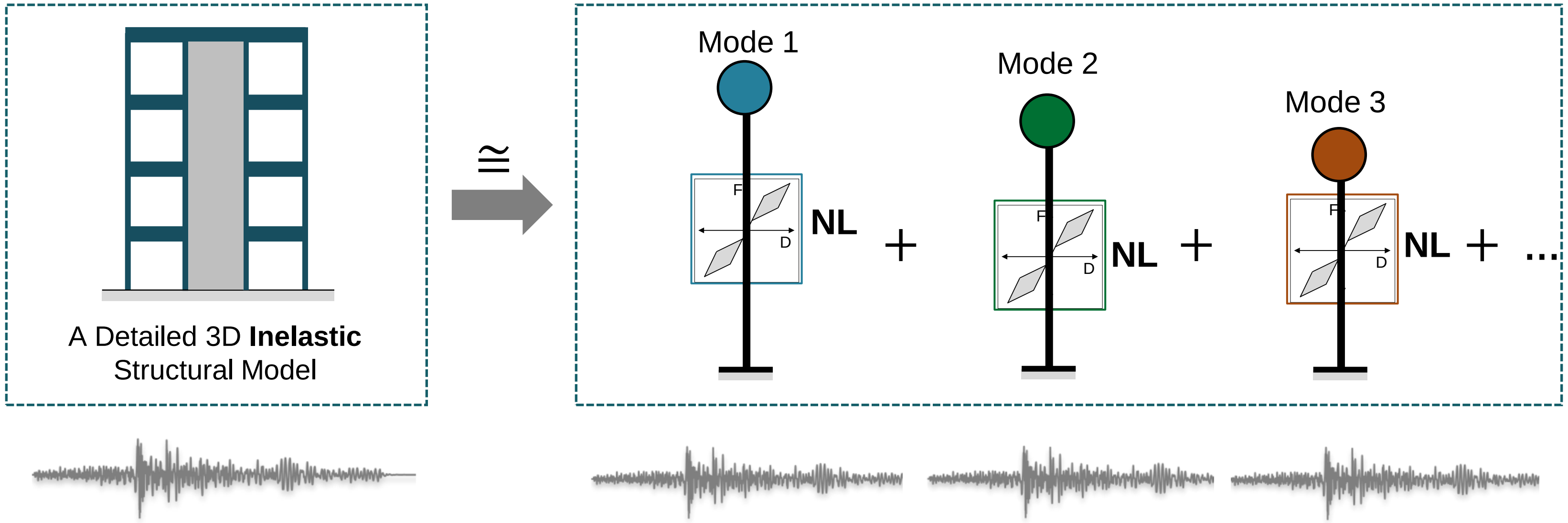
The Uncoupled Modal Response History Analysis Procedure

The UMRHA Procedure

The Classical Modal Analysis Procedure



The Uncoupled Modal Response History Analysis (UMRHA) Procedure



The Uncoupled Modal Response History Analysis (UMRHA) Procedure

Linearly Elastic Systems

- The classical modal analysis procedure for linearly elastic systems is equivalent to finding the response of the structure to $\boldsymbol{P}_{eff,n}(t)$ for each n and superposing the responses for all n.
- The response of the system to $\boldsymbol{P}_{eff,n}(t)$ is entirely in the nth mode, with no contribution from other modes, which implies that the modes are uncoupled.
- The equations governing the response of the linearly elastic MDF system to $\boldsymbol{P}_{eff,n}(t)$,

$$\boldsymbol{M}\ddot{\boldsymbol{u}}(t) + \boldsymbol{C}\dot{\boldsymbol{u}}(t) + \boldsymbol{K}\boldsymbol{u}(t) = -\boldsymbol{s}_n\ddot{u}_g(t)$$

and the resulting floor displacements (using the idea of modal expansion from modal analysis) are

$$\boldsymbol{u}_n(t) = \boldsymbol{\phi}_n q_n(t)$$

Substituting this $\boldsymbol{u}_n(t)$ in governing equation and pre-multiplying by $\boldsymbol{\phi}_{\boldsymbol{n}}^T$ leads to the equation governing the modal coordinate $q_n(t)$:

$$\ddot{q}_n(t) + 2\xi_n\omega_n\dot{q}_n(t) + \omega_n^2\, q_n(t) = -\Gamma_n\,\ddot{u}_g(t)$$

Where ω_n is the natural frequency, ξ_n is the damping ratio, and Γ_n is the modal participation factor for the nth mode.

- As demonstrated in classical modal analysis, the solution of nth-mode equation of motion is

$$q_n(t) = \Gamma_n D_n(t)$$

Where $D_n(t)$ is deformation response of the nth mode linearly elastic SDF system governed by

$$\ddot{D}_n + 2\xi_n \omega_n \dot{D}_n + \omega_n^2 D_n = -\ddot{u}_g(t)$$

Therefore,

$$\boldsymbol{u}_n(t) = \Gamma_n \boldsymbol{\phi}_n D_n(t)$$

$$\Delta_{jn}(t) = \Gamma_n \left(\phi_{jn} - \phi_{j-1,n}\right) D_n(t)$$

- The above equations represent the response of the MDF system to $\boldsymbol{P}_{eff,n}(t)$, and superposing the responses for all n gives the response of the system due to total excitation $\boldsymbol{P}_{eff}(t)$:

$$r(t) = \sum_{n=1}^{N} r_n(t)$$

- The UMRHA procedure for exact analysis of linearly elastic systems is identical to classical modal RHA. But to derive these equations, now we have used the modal expansion of spatial distribution of effective earthquake forces.

The Uncoupled Modal Response History Analysis (UMRHA) Procedure

Inelastic Systems

- Although modal analysis is not valid for an inelastic system, its dynamic response can usefully be discussed in terms of the natural vibration modes of the corresponding linear system.

- Each structural element of this linear system is defined to have the same stiffness as its initial stiffness in the inelastic system; both systems have the same mass and damping. Therefore, the natural vibration periods and modes of the corresponding linear system are the same as the vibration properties of the inelastic system undergoing small oscillation, which are referred to as “periods” and “modes” of the inelastic system.

- Thus, the modal expansion of effective earthquake forces is also valid for inelastic systems, where ϕ_n now represents the modes of the corresponding linear system.

The Uncoupled Modal Response History Analysis (UMRHA) Procedure

Inelastic Systems

- The equations governing the response of the inelastic MDF system to $\boldsymbol{P}_{eff,n}(t)$ are

$$\boldsymbol{M}\ddot{\boldsymbol{u}}(t) + \boldsymbol{C}\dot{\boldsymbol{u}}(t) + \boldsymbol{f}_{\boldsymbol{s}}(\boldsymbol{u}) = -\boldsymbol{s}_n \ddot{u}_g(t)$$

- The solution of this quation will no longer be described by $\boldsymbol{u}_n(t) = \boldsymbol{\phi}_n q_n(t)$ because modes other than the nth mode will also contribute to the system response, implying that the vibration modes are now coupled.
- Thus, the floor displacements are given by:

$$\boldsymbol{u}_n(t) = \sum_{r=1}^{N} \boldsymbol{\phi}_r q_r(t)$$

- However, because for linear systems $q_r(t) = 0$ for all modes other than the *n*th mode, it is reasonable to expect that $q_r(t)$ may be small for inelastic systems, implying that the elastic modes are, at most, weakly coupled. Therefore,

$$\boldsymbol{u}_n(t) = \sum_{r=1}^{N} \boldsymbol{\phi}_r q_r(t) \simeq \boldsymbol{\phi}_n q_n(t)$$

Figure shows that the roof displacement due to the force vector $\boldsymbol{P}_{eff,n}(t)$ is due primarily to the *n*th mode but that other modes contribute to the response. The second, third, and fourth modes start responding to excitation $\boldsymbol{P}_{eff,1}(t)$ the instant the structure first yields.

Although the natural vibration modes are no longer uncoupled if the system responds in the inelastic range, modal coupling is weak.

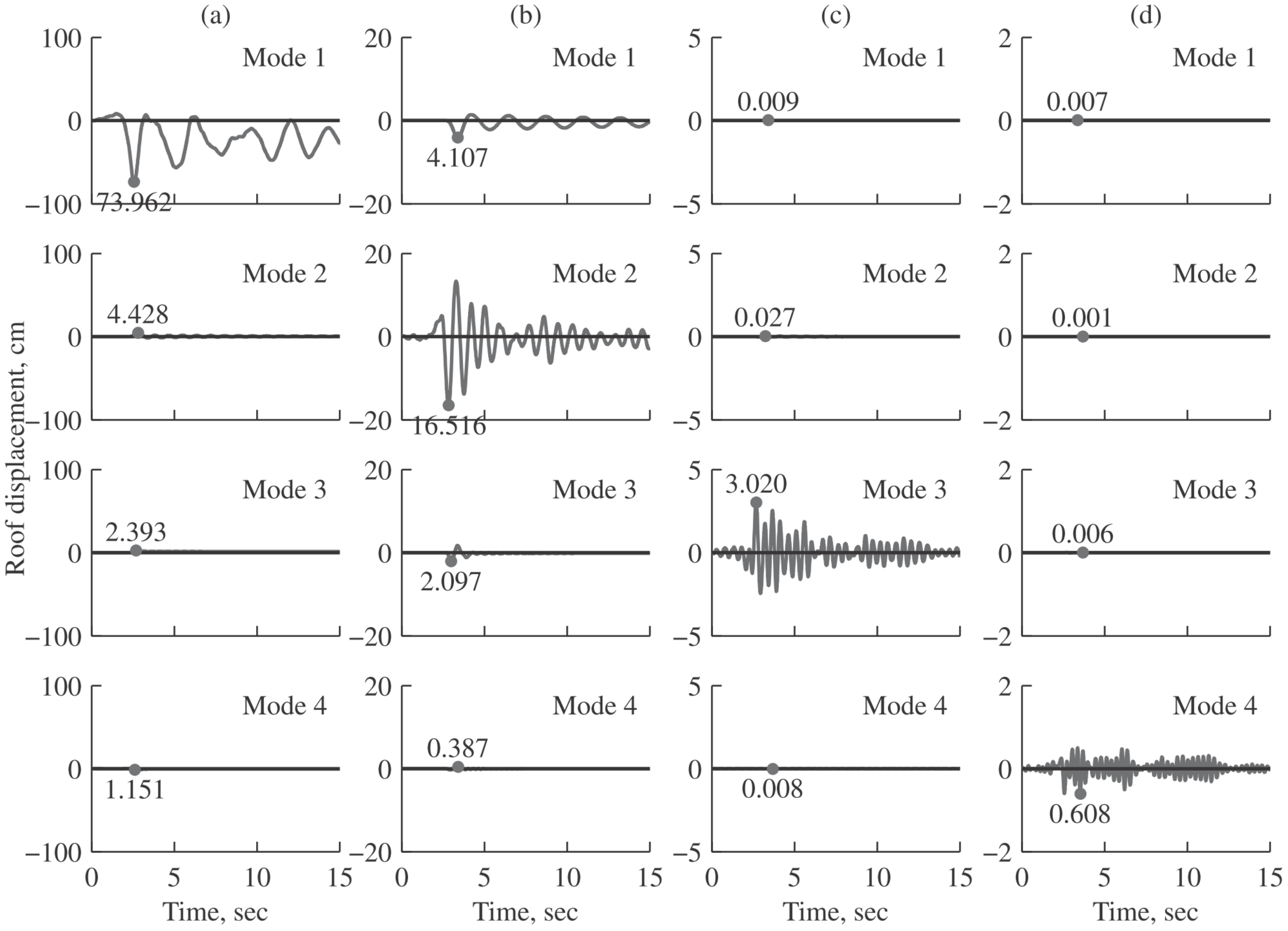


Figure 20.6.1 Modal decomposition of roof displacement due to $\mathbf{p}_{\text{eff},n}(t) = -\mathbf{s}_n \ddot{u}_g(t)$, $n = 1, 2, 3$, and 4, where $\ddot{u}_g(t)$ = LA25 ground motion: (a) $\mathbf{p}_{\text{eff},1} = -\mathbf{s}_1 \times$ LA25; (b) $\mathbf{p}_{\text{eff},2} = -\mathbf{s}_2 \times$ LA25; (c) $\mathbf{p}_{\text{eff},3} = -\mathbf{s}_3 \times$ LA25; (d) $\mathbf{p}_{\text{eff},4} = -\mathbf{s}_4 \times$ LA25.

- This weak coupling of modes implies that the structural response due to excitation $\boldsymbol{P}_{eff,n}(t)$ may be approximated b

$$\boldsymbol{u}_n(t) \simeq \boldsymbol{\phi}_n q_n(t)$$

- Substituting this approximation into governing equation and pre-multiplying by $\boldsymbol{\phi}_n^T$ gives

$$\ddot{q}_n(t) + 2\xi_n\omega_n\dot{q}_n(t) + \frac{F_{sn}}{M_n} = -\Gamma_n\ddot{u}_g(t)$$

where F_{sn} is a nonlinear hysteretic function of the nth modal coordinate q_n:

$$F_{sn} = F_{sn}(q_n) = \phi_n^T \boldsymbol{f}_s(q_n)$$

- If the smaller contributions of other modes had not been neglected, F_{sn} would depend on all modal coordinates, and the set of equations would be coupled because of yielding of the structure.
- By comparing the nth-mode governing equation of motion with linear counterpart, the solution can be expressed as $q_n(t) = \Gamma_n D_n(t)$, where $D_n(t)$ is now governed by

$$\ddot{D}_n(t) + 2\xi_n\omega_n\dot{D}_n(t) + \frac{F_{sn}}{L_n} = -\ddot{u}_g(t)$$

- D_n may be interpreted as the deformation response of the nth-mode inelastic SDF system, an SDF defined by
  1) small-oscillation vibration properties—natural frequency ω_n (natural period T_n) and damping ratio ξ_n—of the nth mode of the MDF system; and
  2) The force–deformation ($F_{sn}/L_n - D_n$) relation. Introducing the nth-mode inelastic SDF system permitted the extension to inelastic systems of the well-established concepts for elastic systems.
- The solution of the nonlinear modal equation provides $D_n(t)$, which can be substituted into following (same) equations to obtain floor displacements and story drifts.

$$\boldsymbol{u}_n(t) = \Gamma_n \boldsymbol{\phi}_n D_n(t)$$

$$\Delta_{jn}(t) = \Gamma_n \left(\phi_{jn} - \phi_{j-1,n}\right) D_n(t)$$

- These equations approximate the response of the inelastic MDF system to $\boldsymbol{P}_{eff,n}(t)$, the nth-mode contribution to $\boldsymbol{P}_{eff}(t)$.
- Superposition of responses to $\boldsymbol{P}_{eff,n}(t)$—n = 1 , 2 , . . . , N—provides the total response to $\boldsymbol{P}_{eff}(t)$.

$$r(t) = \sum_{n=1}^{N} r_n(t)$$

- This is the UMRHA procedure for approximate analysis of inelastic systems.

To test the modal uncoupling approximation in UMRHA, the response of the a 9-story building to $\boldsymbol{P}_{eff,n}(t) = -\boldsymbol{s}_n \ddot{u}_g(t)$ is determined by two methods and compared.

Left: Roof Displacement

Right: Roof Drift

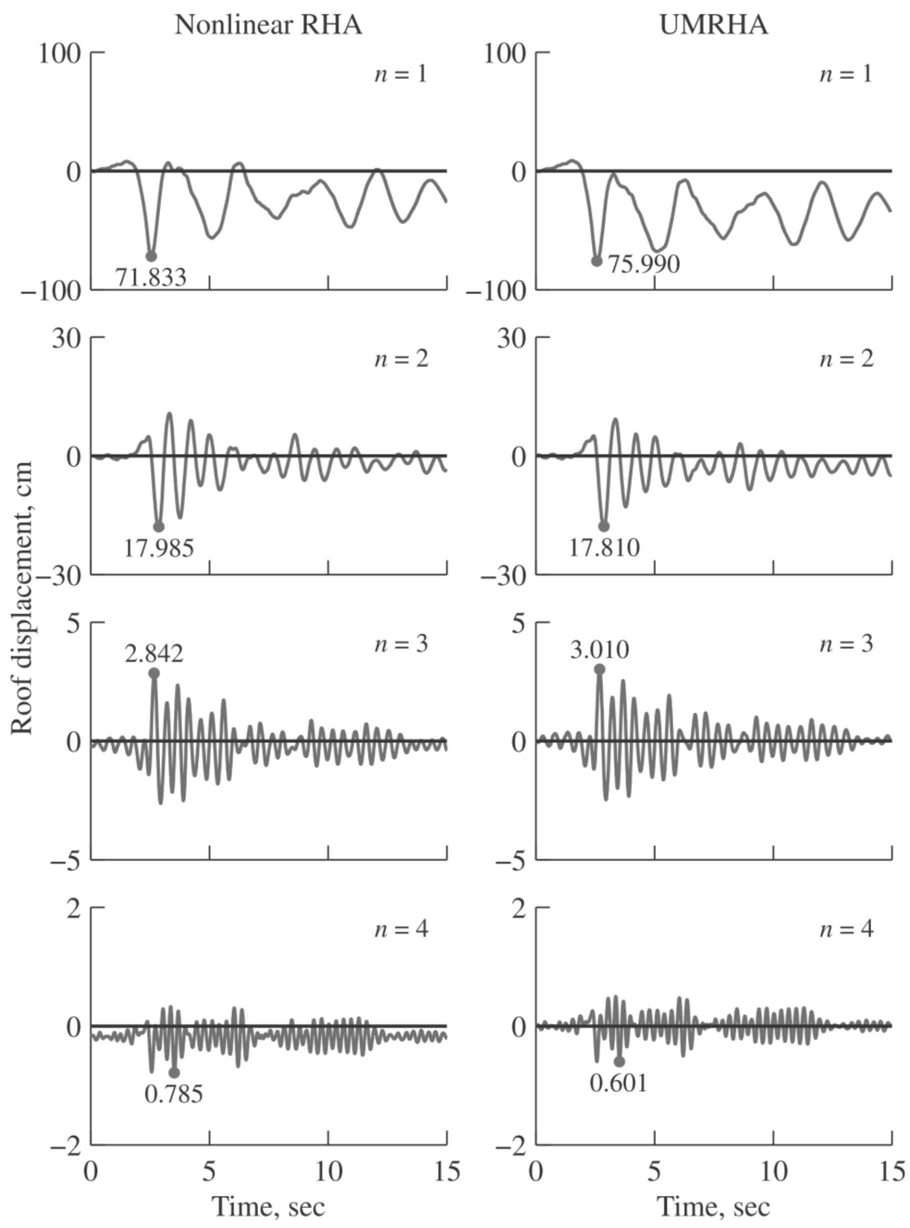


Figure 20.6.2 Comparison of approximate roof displacement from UMRHA and exact result from nonlinear RHA due to $\mathbf{p}_{\text{eff},n}(t) = -\mathbf{s}_n \ddot{u}_g(t)$, $n = 1, 2, 3,$ and 4, where $\ddot{u}_g(t) =$ LA25 ground motion.

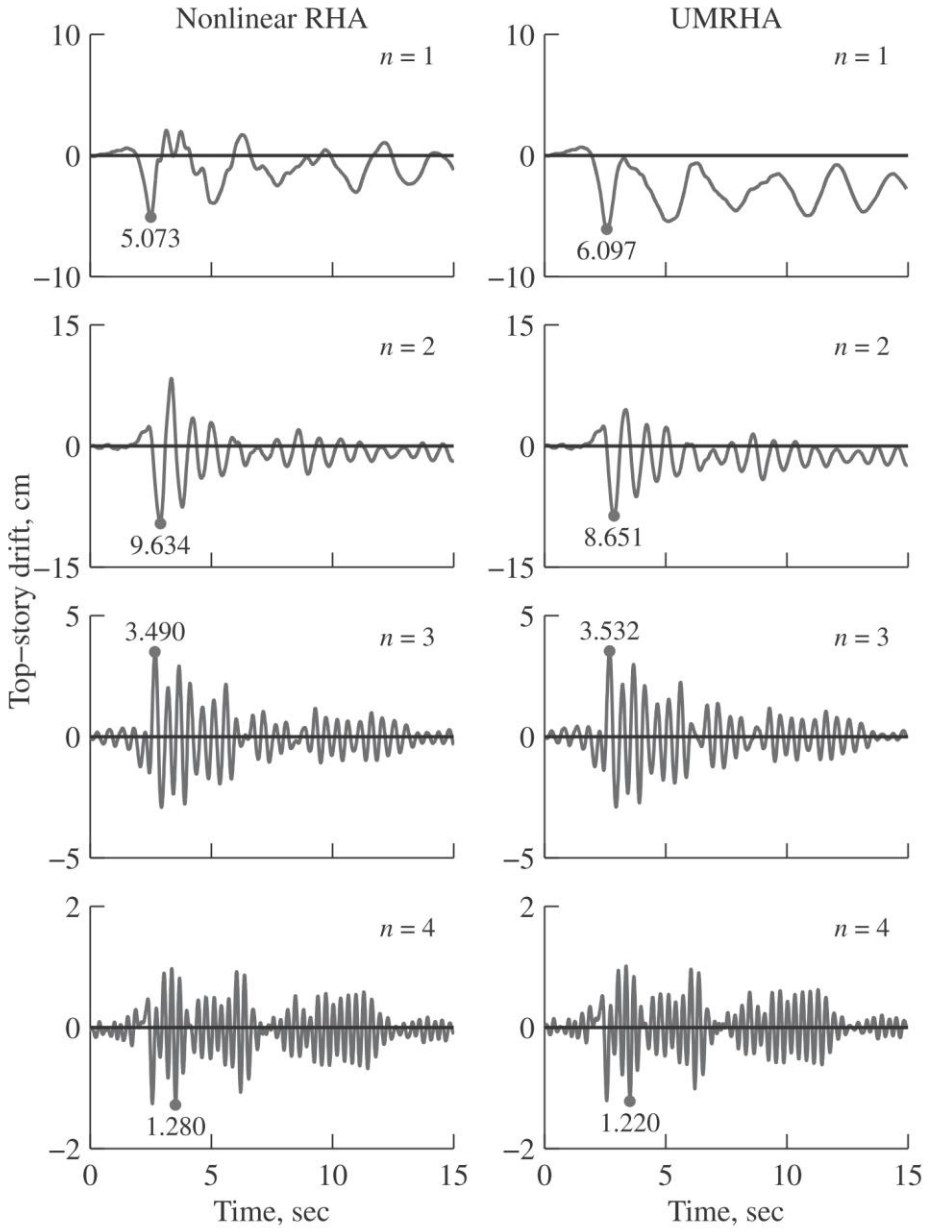
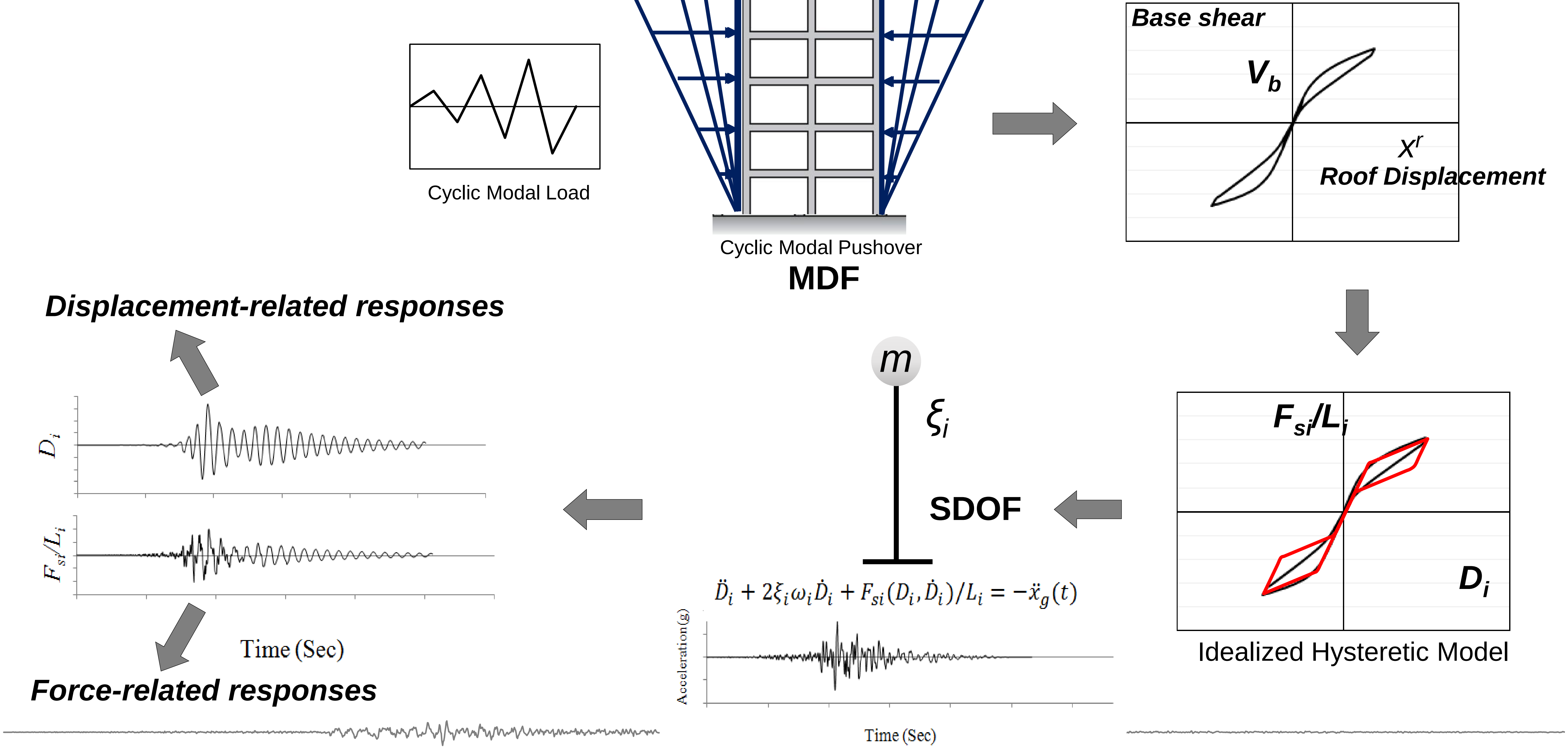


Figure 20.6.3 Comparison of approximate top-story drift from UMRHA and exact result from nonlinear RHA due to $\mathbf{p}_{\text{eff},n}(t) = -\mathbf{s}_n \ddot{u}_g(t)$, $n = 1, 2, 3,$ and 4, where $\ddot{u}_g(t) =$ LA25 ground motion.

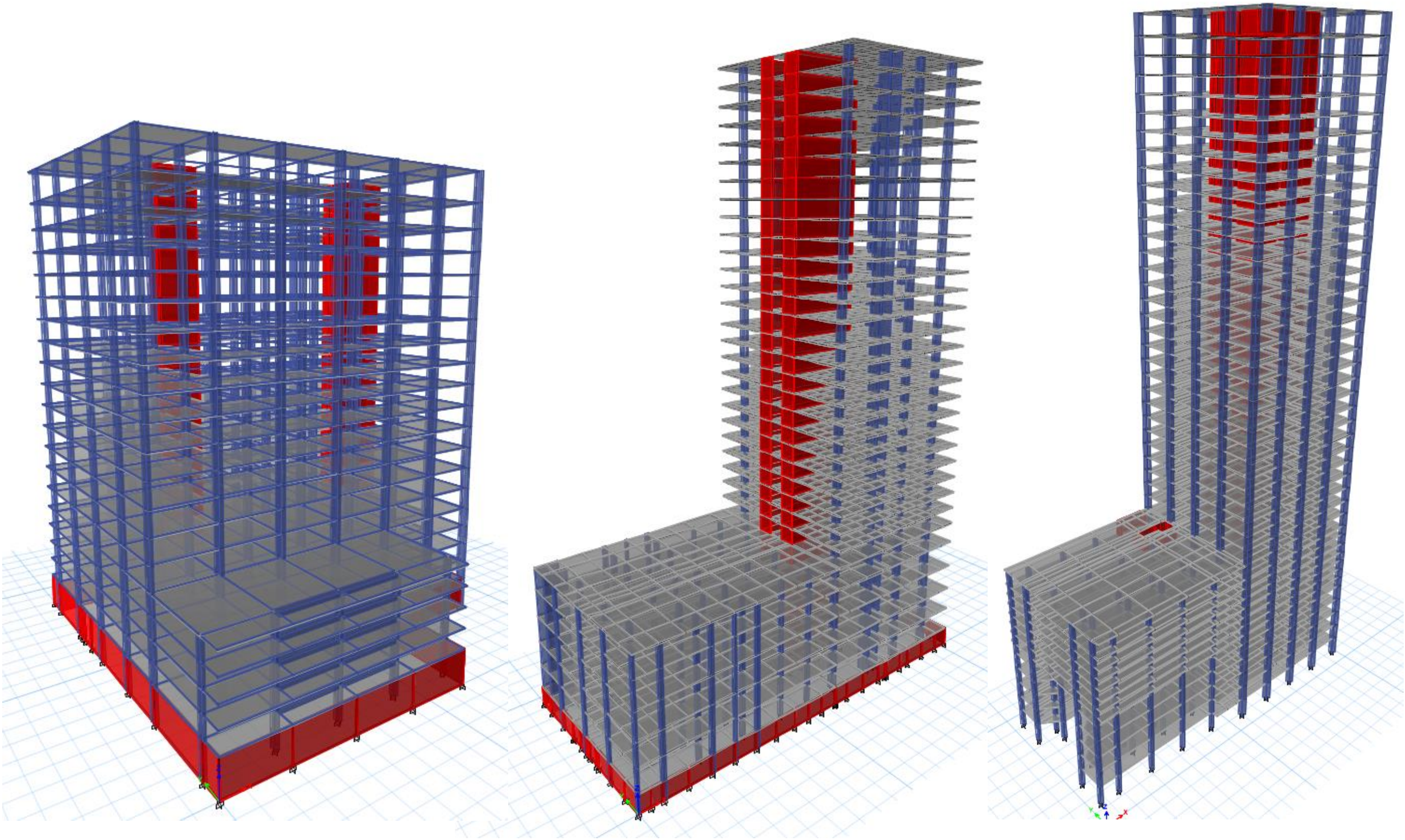
The UMRHA Procedure



The Uncoupled Modal Response History Analysis (UMRHA) Procedure

- For elastic systems → UMRHA = Classical Modal RHA (an exact analysis procedure).
- For inelastic systems → UMRHA = Approximate analysis procedure.
- The UMRHA for inelastic systems is based on two approximations
  1) Superposition of responses (Strictly valid for only elastic systems. Approximately valid for inelastic systems)
  2) Neglecting the coupling of modal coordinates, which permitted computing the response of inelastic MDF system to $\boldsymbol{P}_{eff,n}(t)$ from that of an SDF system. This approximation is reasonable only because the excitation is $\boldsymbol{P}_{eff,n}(t)$, the nth-mode contribution to the total excitation $\boldsymbol{P}_{eff}(t)$. It would not be valid for an excitation with lateral force distribution different than $\boldsymbol{s_n}$ [e.g., the total excitation $\boldsymbol{P}_{eff}(t)$], pointing out that the modal expansion of effective earthquake forces is a key concept underlying the UMRHA.

Case study Buildings



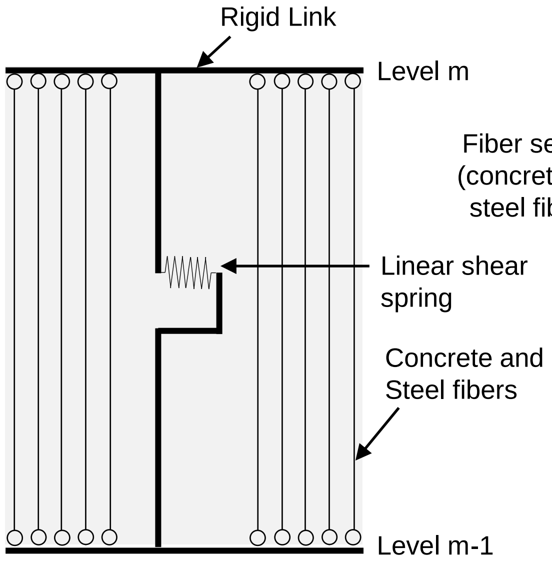
20-story
B1

33-story
B2

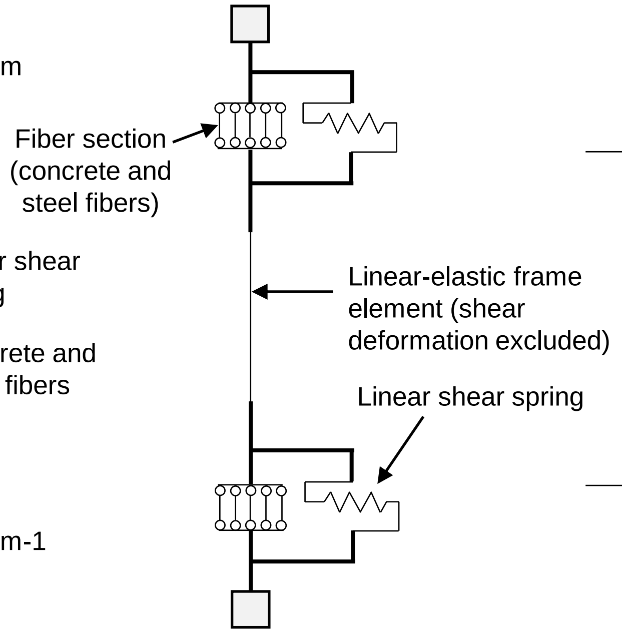
44-story
B3

- Located in Bangkok, Thailand
- Heights vary from 20 to 44 stories
- RC slab-column frames carry gravity loads
- RC walls & cores resist lateral loads
- Masonry infill walls extensively used
- Designed for wind loads, but not for seismic effects
- Possess irregular features commonly found in typical tall buildings, e.g. podium and non-symmetrical arrangement of RC walls, etc.

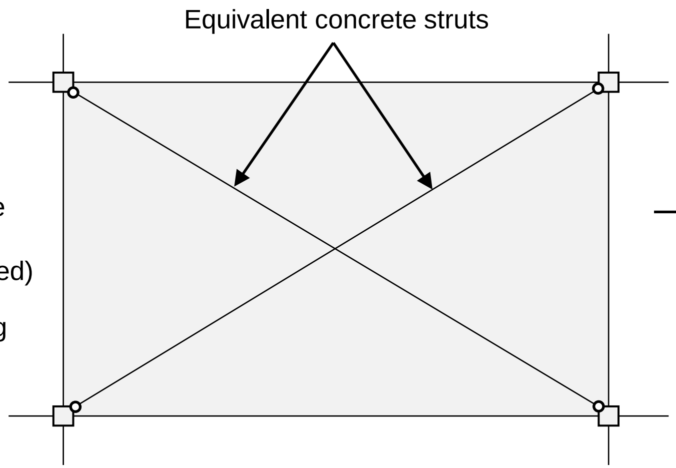
Nonlinear Modeling of Case Study Buildings



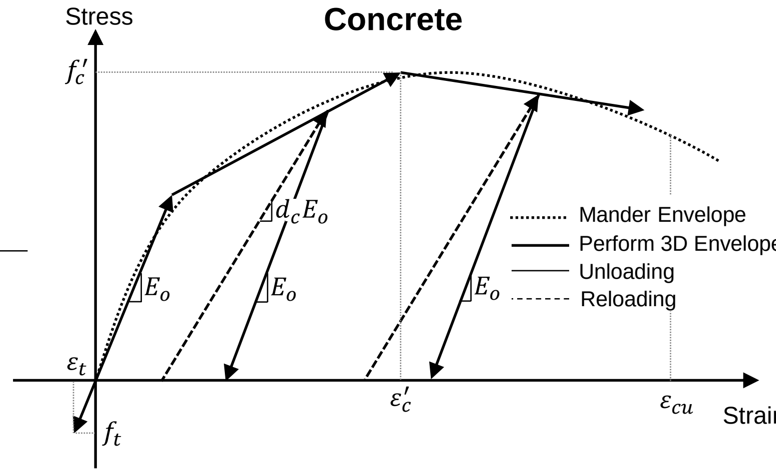
MVLEM for RC Walls

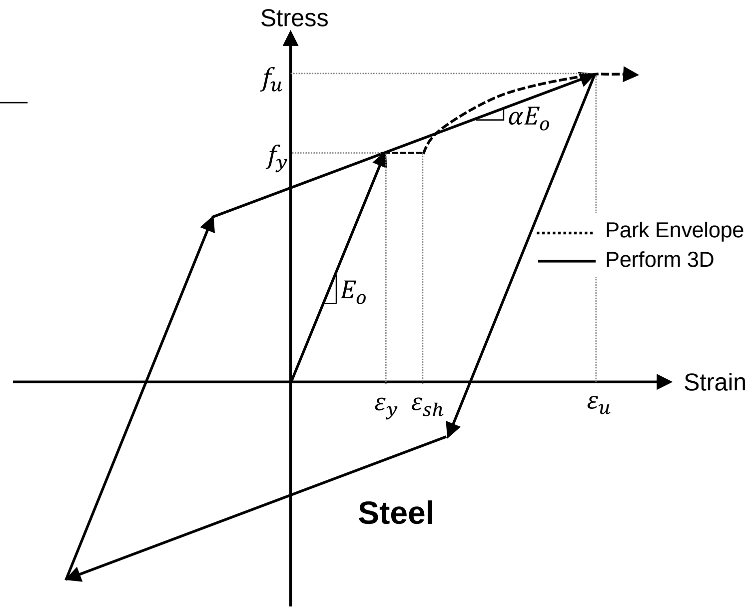


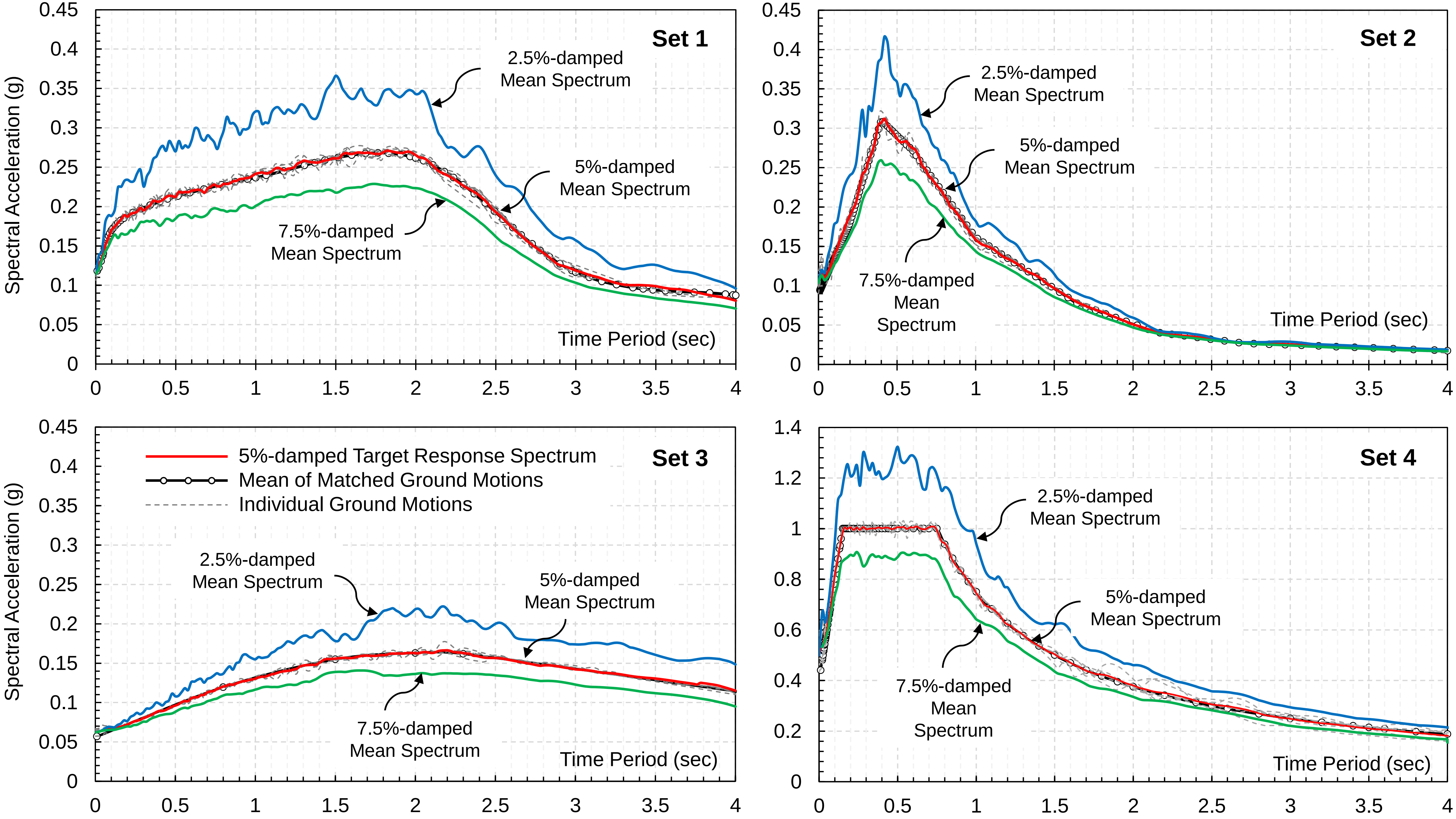
Lumped Fibers for RC Columns



Masonry Infill Wall Model

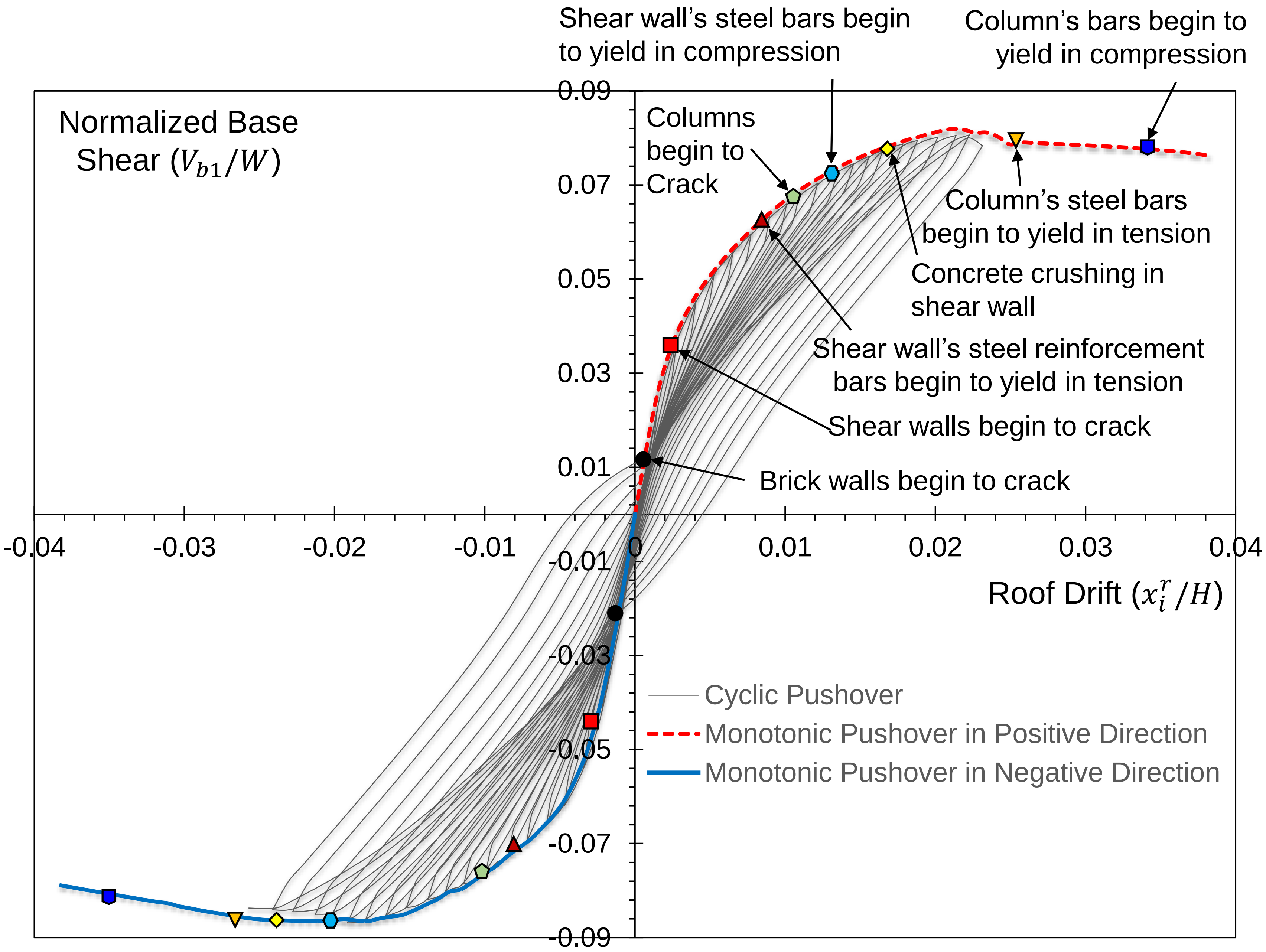




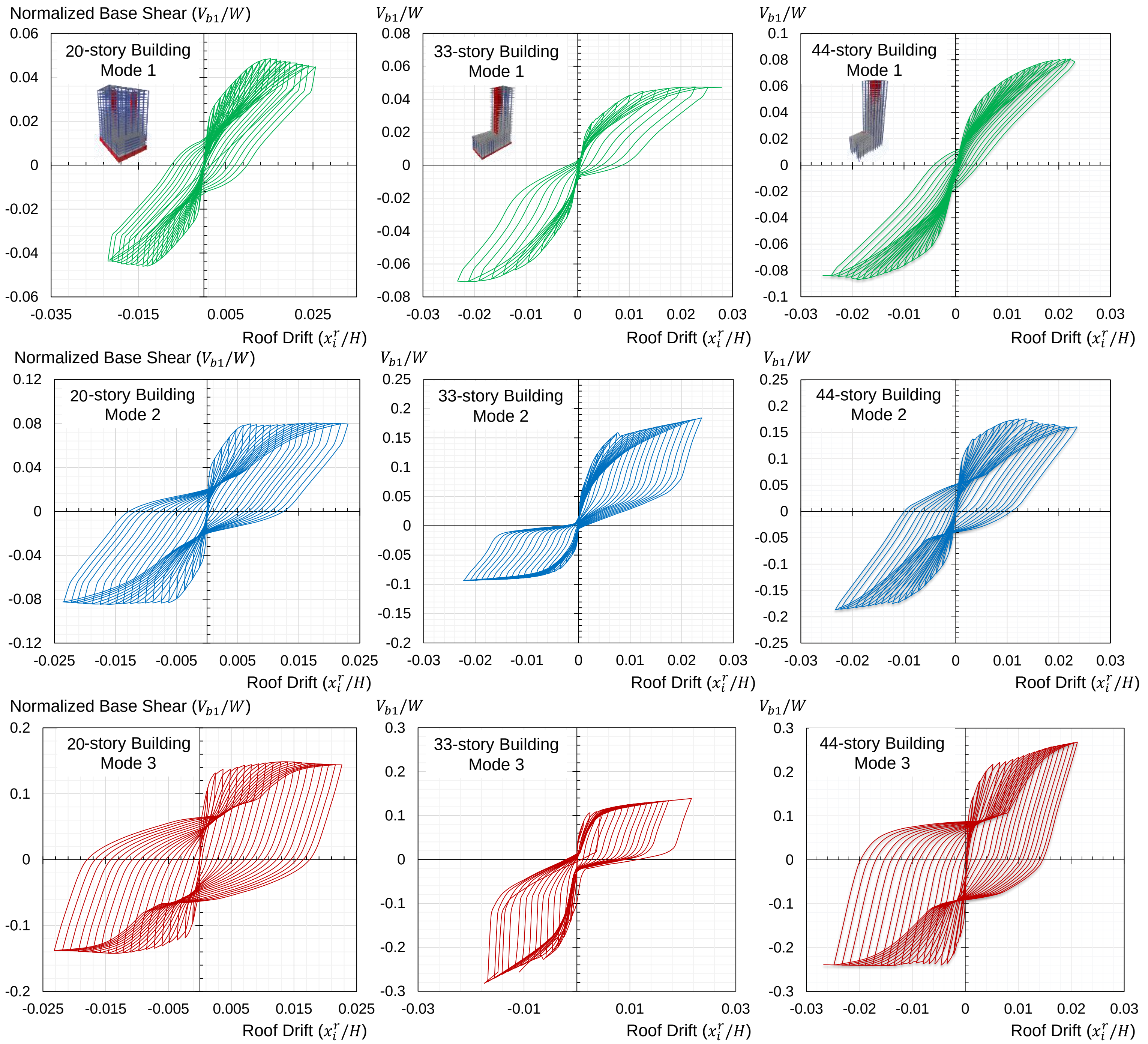

Set 1
2.5%-damped Mean Spectrum
5%-damped Mean Spectrum
7.5%-damped Mean Spectrum
Spectral Acceleration (g)
Time Period (sec)
Set 2
2.5%-damped Mean Spectrum
5%-damped Mean Spectrum
7.5%-damped Mean Spectrum
Time Period (sec)
Set 3
5%-damped Target Response Spectrum
Mean of Matched Ground Motions
Individual Ground Motions
2.5%-damped Mean Spectrum
5%-damped Mean Spectrum
7.5%-damped Mean Spectrum
Spectral Acceleration (g)
Time Period (sec)
Set 4
2.5%-damped Mean Spectrum
5%-damped Mean Spectrum
7.5%-damped Mean Spectrum
Time Period (sec)

The Cyclic Behavior of Case Study Buildings

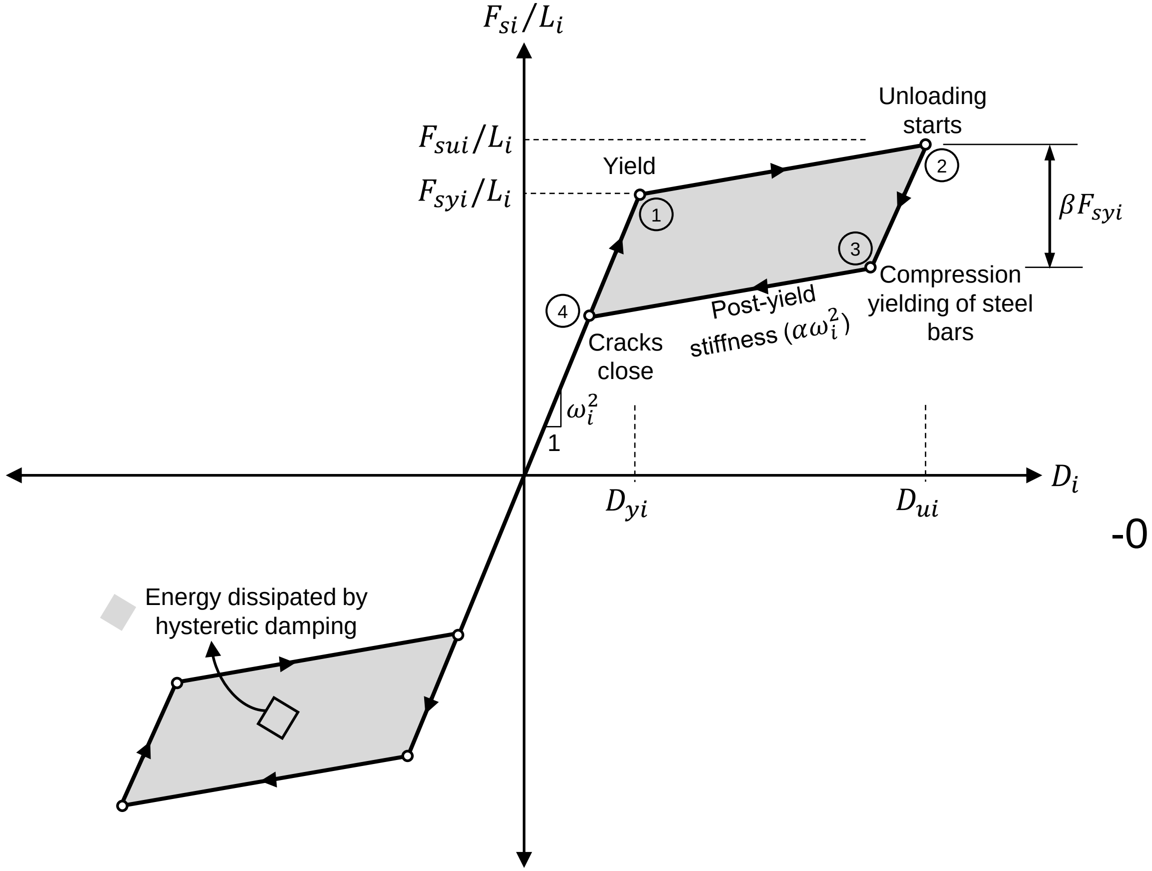
44-story case study building in Strong Direction

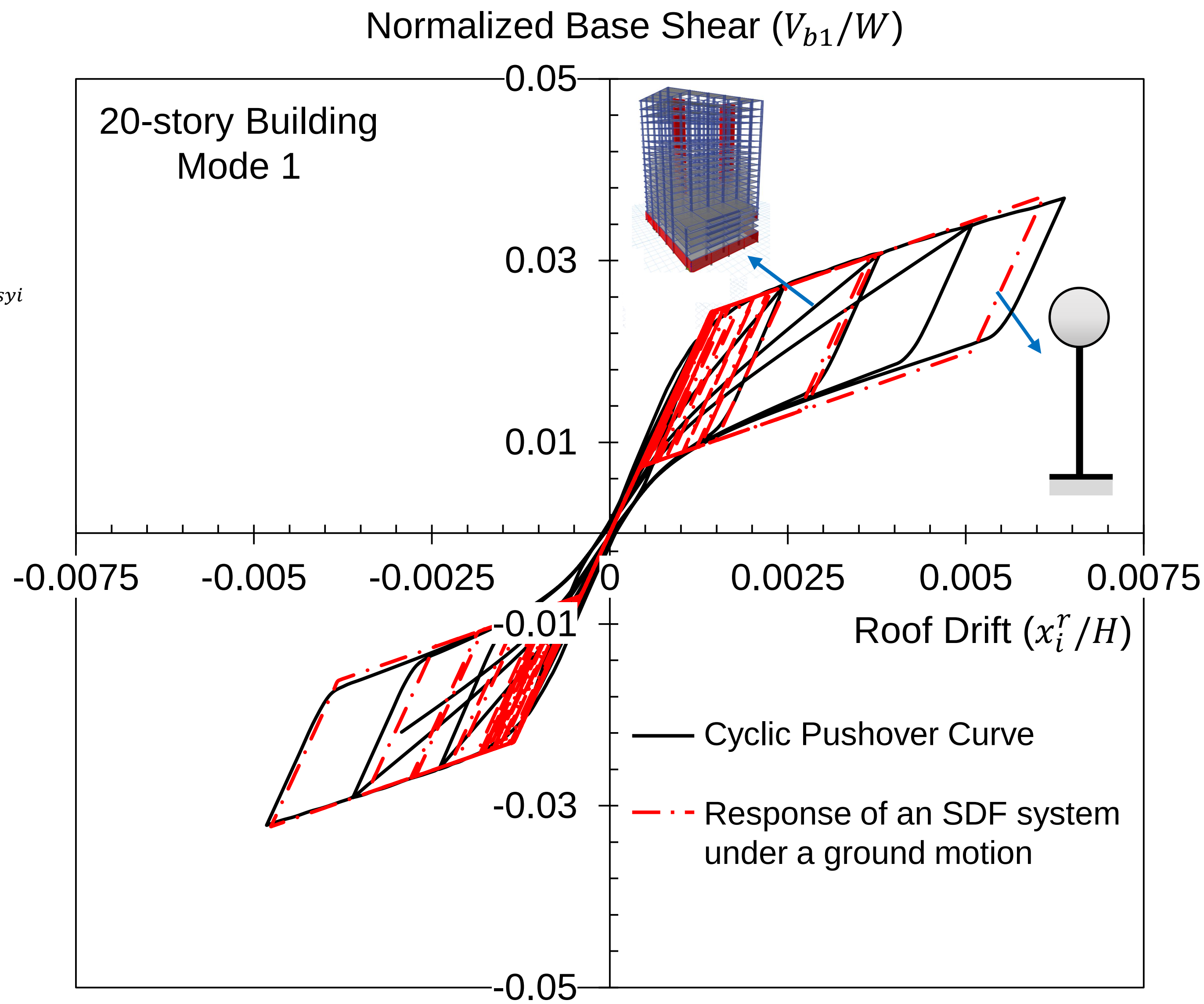


The Cyclic Behavior of Case Study Buildings

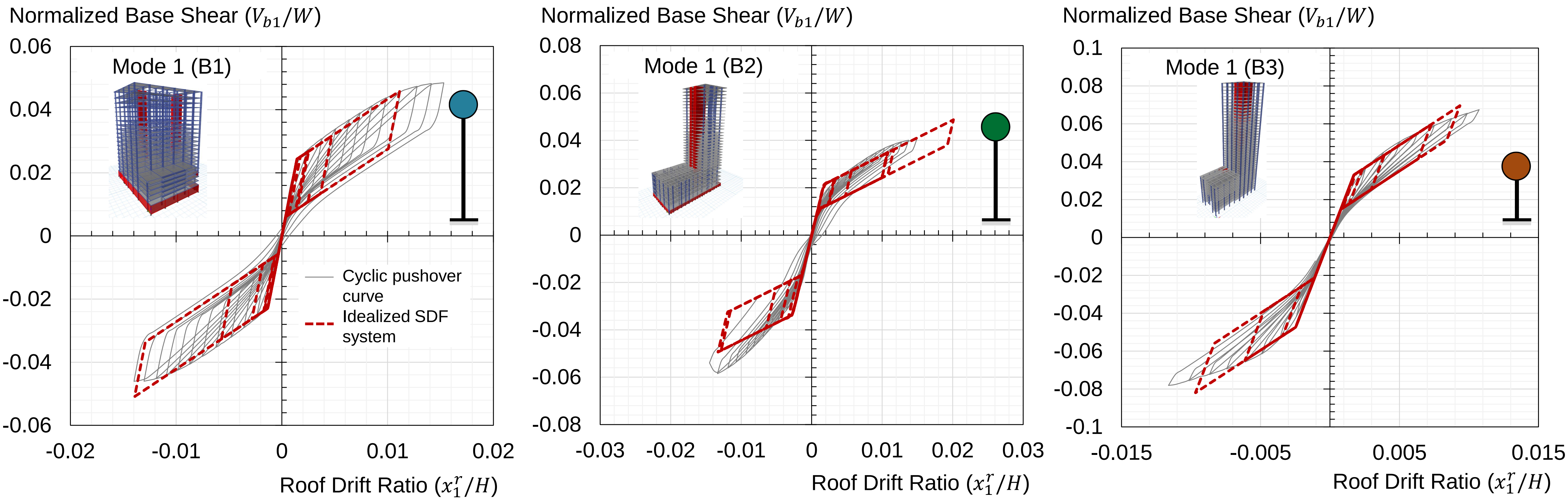


Idealization of Cyclic Pushover Curves



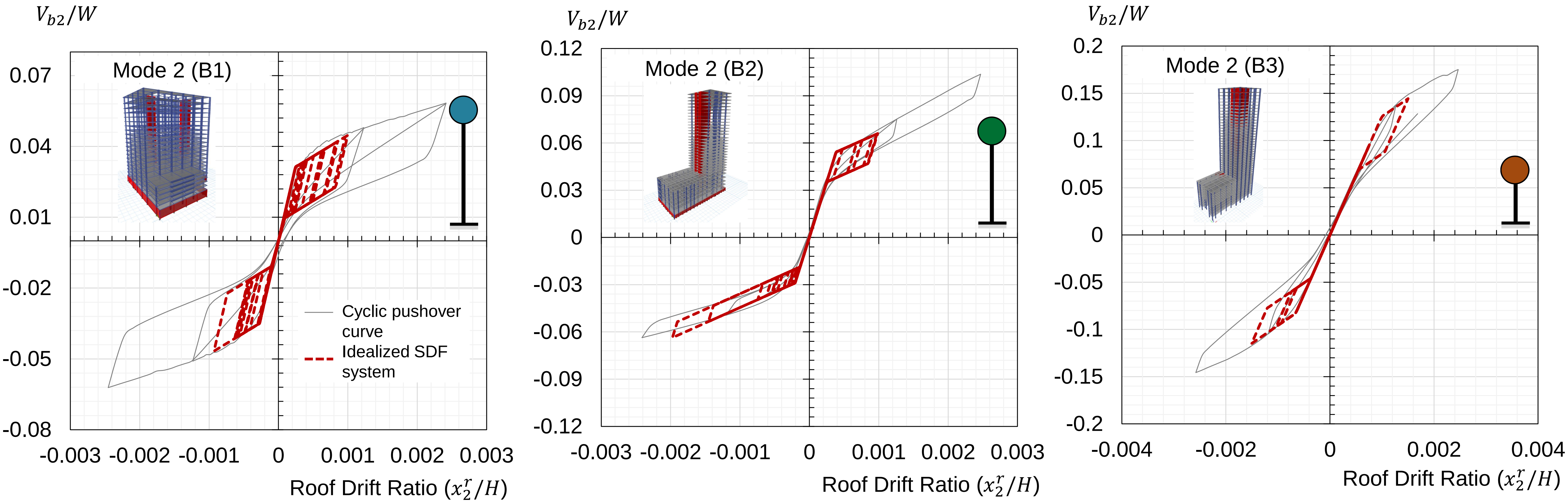


Idealization of Cyclic Pushover Curves



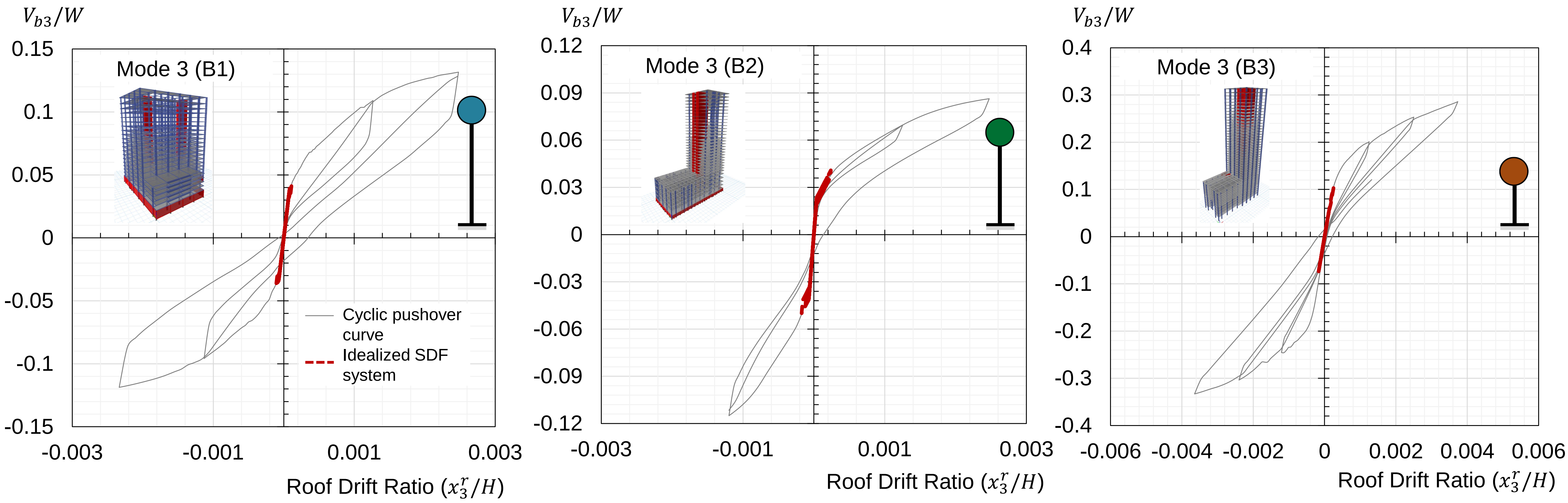
Ground Motion Set 4

Idealization of Cyclic Pushover Curves



Ground Motion Set 4

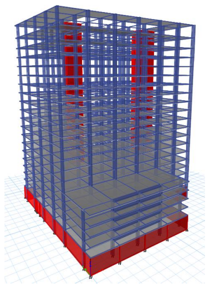
Idealization of Cyclic Pushover Curves

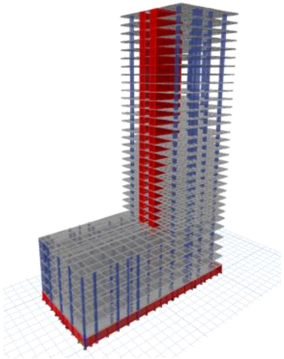


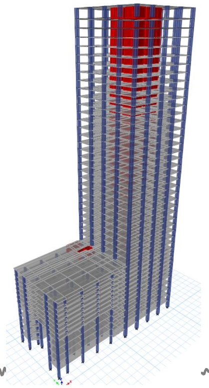
Ground Motion Set 4

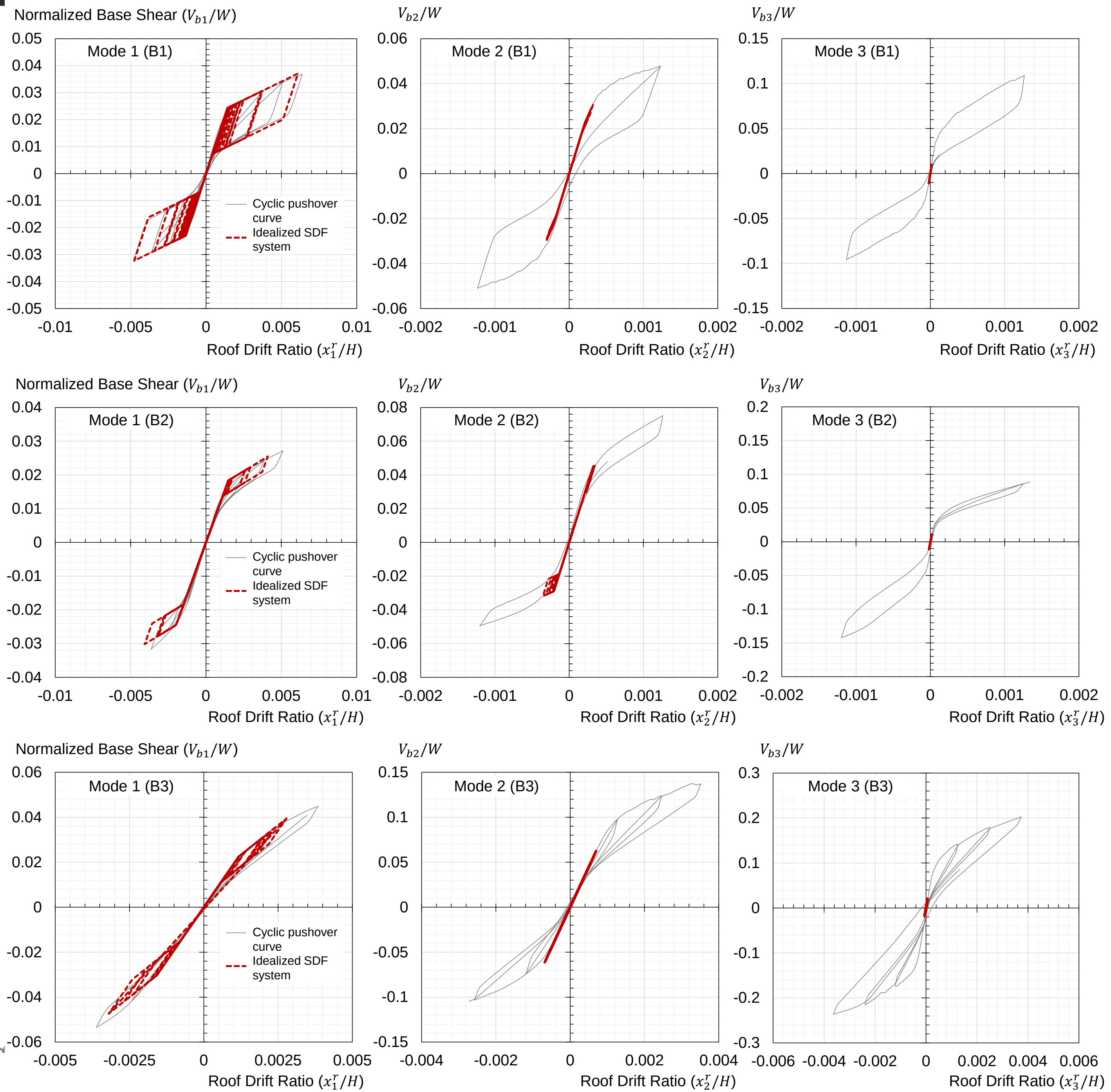
The Idealization of Cyclic Pushover Curves

Ground Motion Set 1





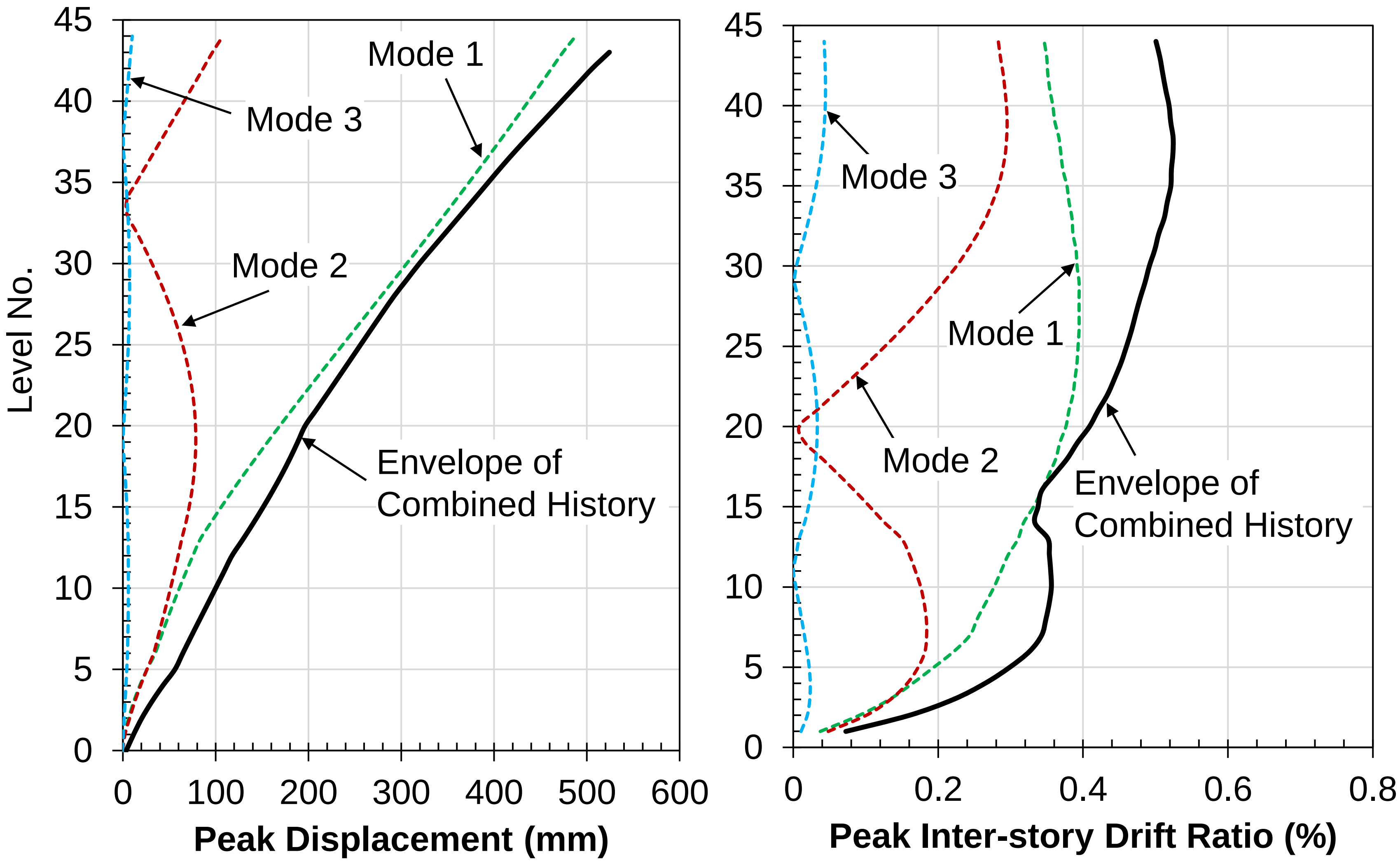




Modal Decomposition of Nonlinear Responses

44-story case study building in Strong Direction

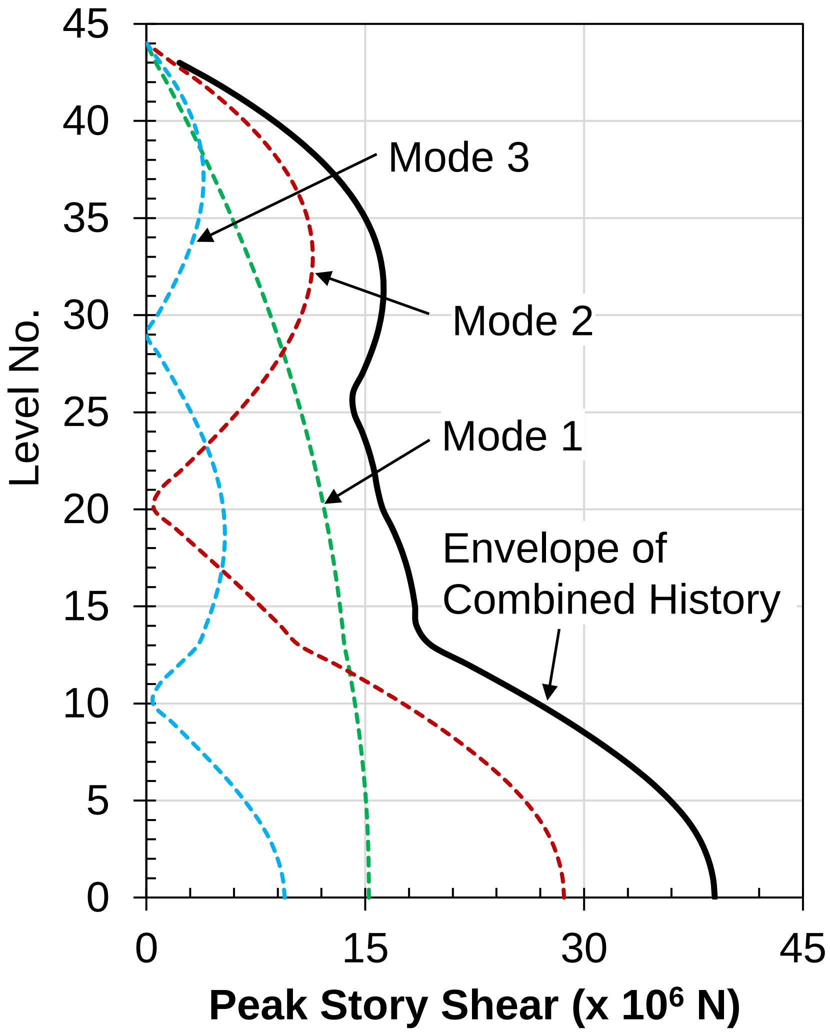
Ground Motion Set 4

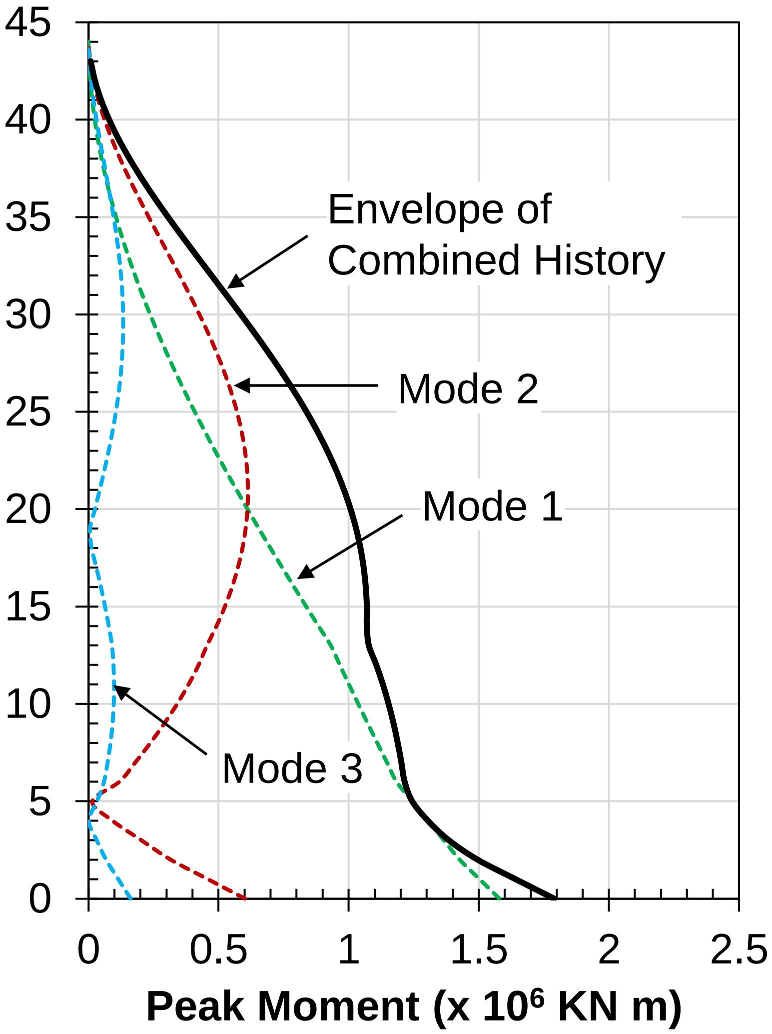


Modal Decomposition of Nonlinear Responses

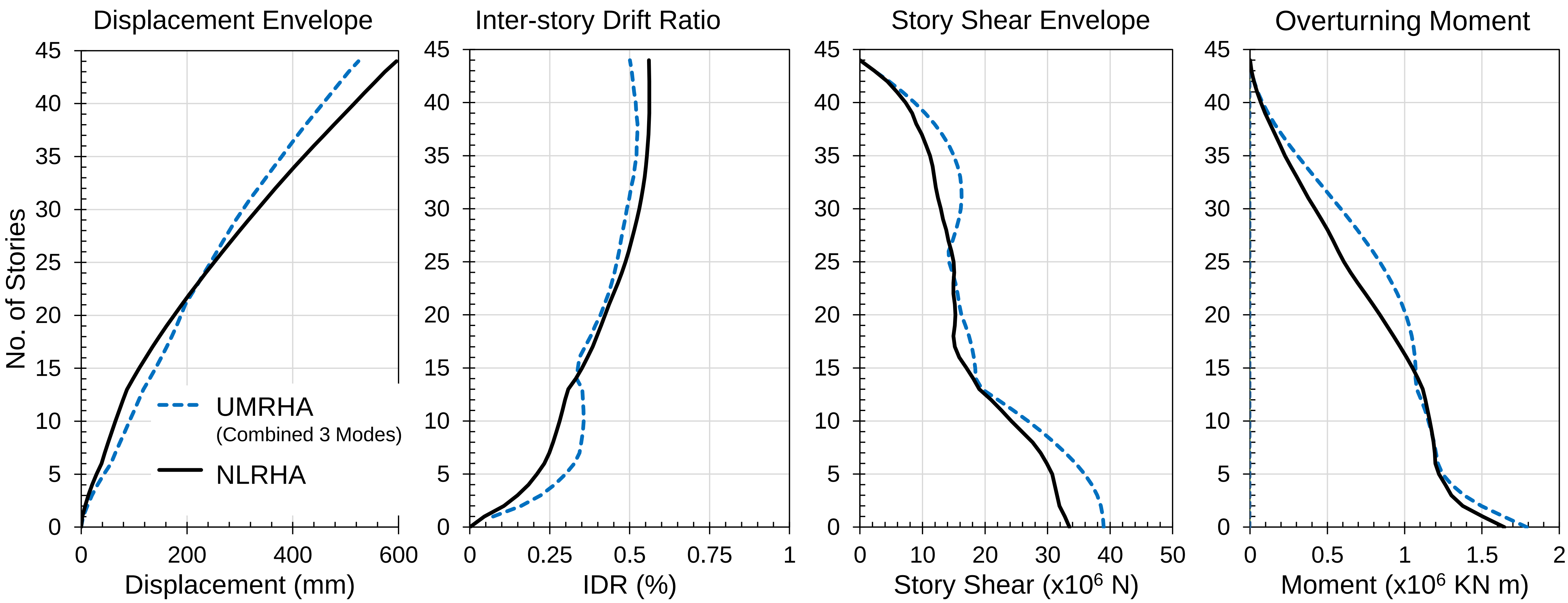
44-story case study building in Strong Direction

Ground Motion Set 4





UMRHA vs. NLRHA

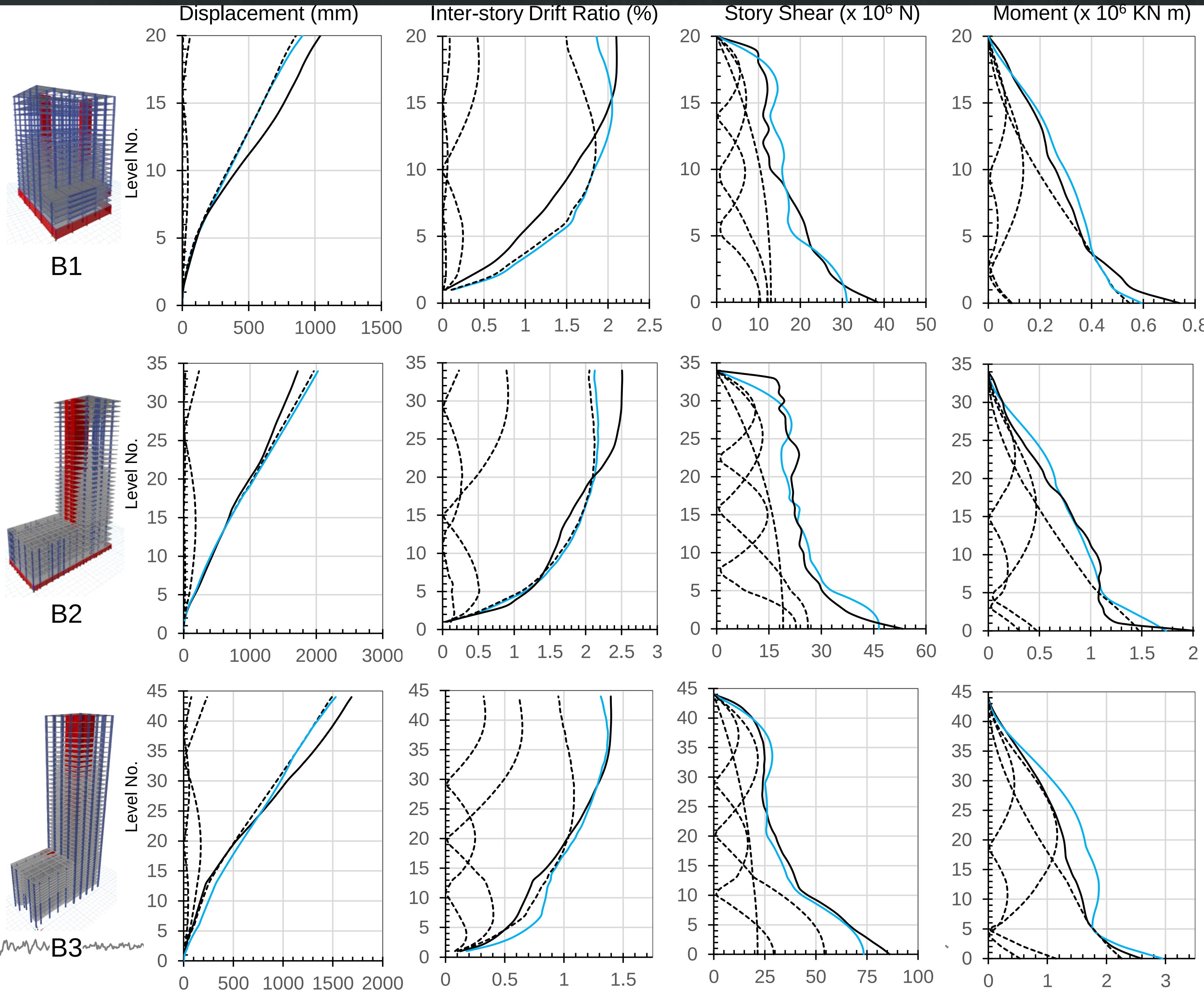


44-story case study building in Strong Direction - Ground Motion Set 4

Modal Decomposition of Nonlinear Responses

44-story case study building in Strong Direction

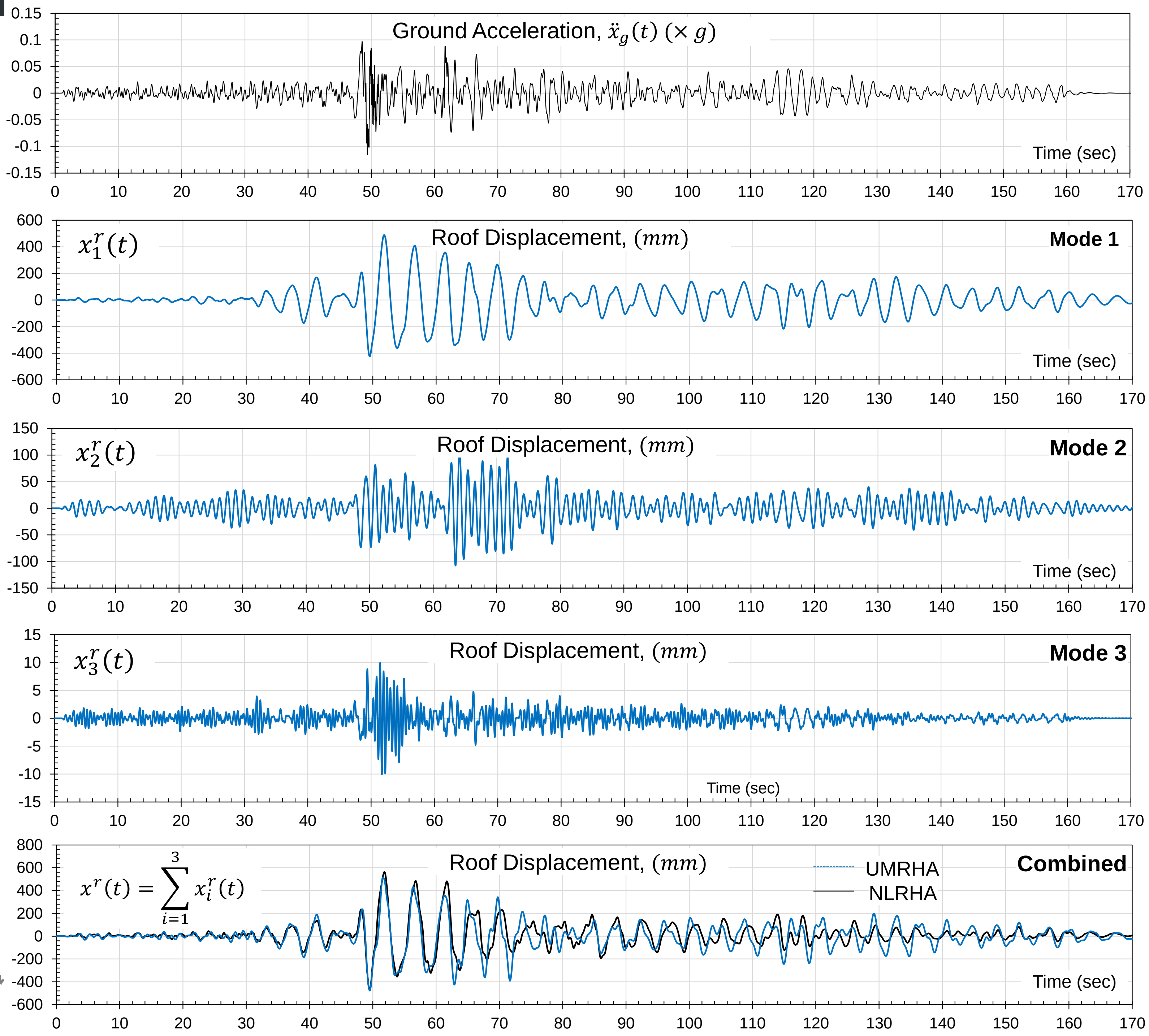
Ground Motion Set 4



Modal Decomposition of Nonlinear Responses

44-story case study building in Strong Direction

Ground Motion Set 4

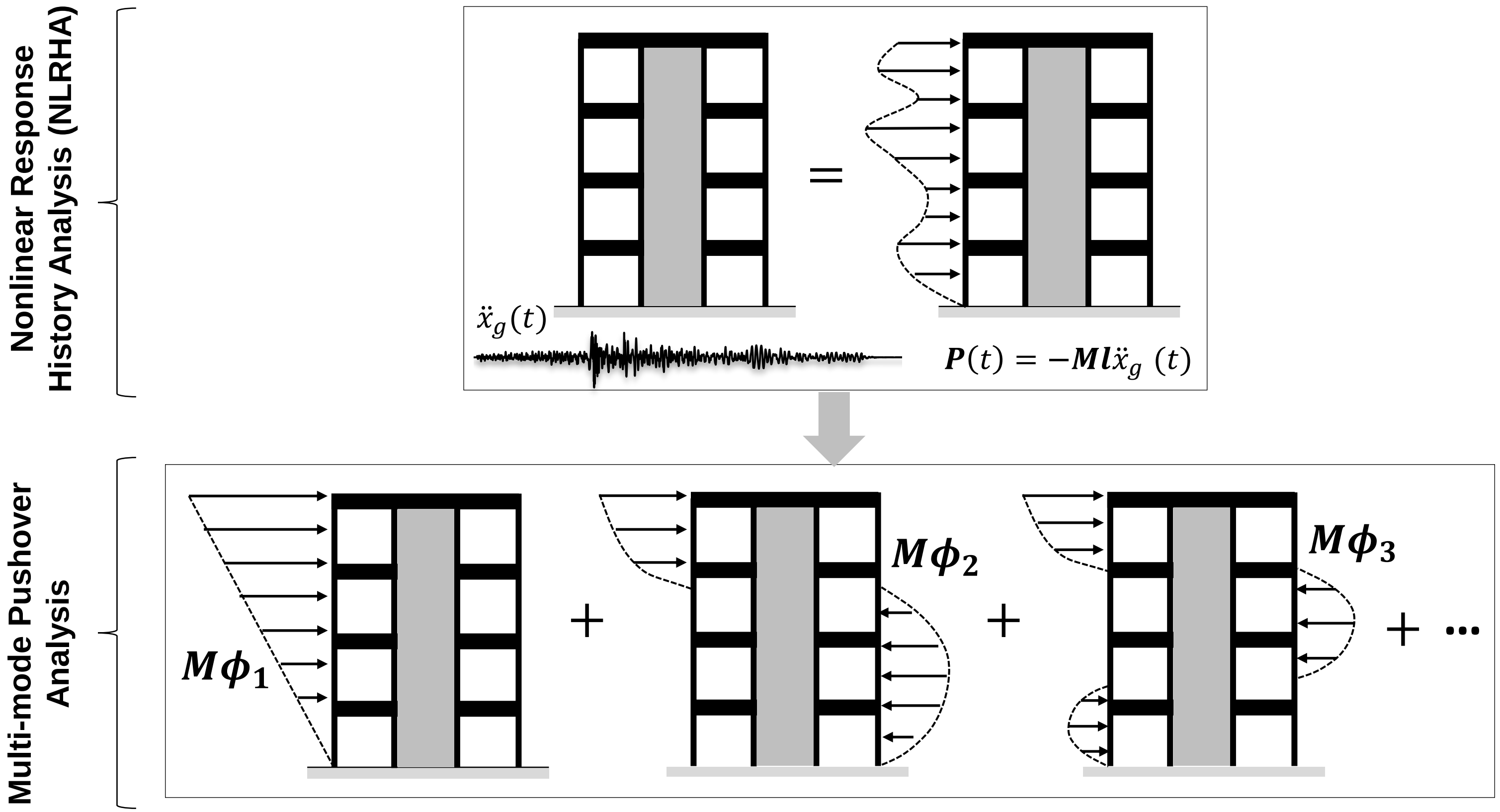


Conclusions

- UMRHA + Accurate Modal Hysteretic Model is able to compute non-linear seismic responses of RC tall buildings with reasonable accuracy.
- The required computational effort is very low compared to that of NLRHA.
- More understanding in complex non-linear dynamic responses of tall buildings can be gained by ‘Modal Decomposition’ of responses.
- This allows engineers to develop effective strategies to improve the seismic performance of these buildings.

The Modal Pushover Analysis (MPA) Procedure (Chopra and Goel, 2002)

The Modal Pushover Analysis Procedure (MPA)



The Modal Pushover Analysis Procedure (MPA)

Linearly Elastic Systems

- The response spectrum analysis (RSA) procedure, which is a dynamic analysis procedure, can be interpreted in two ways: as static analysis or as pushover analysis.
  a) Static analysis of the building subjected to lateral forces $\boldsymbol{f}_n = \boldsymbol{s}_n A_n = \Gamma_n \boldsymbol{M} \boldsymbol{\phi}_n A_n$ will provide the same value of r_n, the peak value of the n^{th}-mode response $r_n(t)$, as obtained from the RSA procedure. (Where $A_n = A(T_n, \xi_n)$, the pseudo-acceleration spectrum ordinate corresponding to the natural vibration period T_n and damping ratio ξ_n of the *n*th mode).
  b) Alternatively, this peak modal response can be obtained by linear static analysis of the structure subjected to monotonically increasing lateral forces with an invariant height-wise distribution: $\boldsymbol{s}_n^* = \boldsymbol{M}\,\boldsymbol{\phi}_n$, pushing the structure up to the roof displacement, u_{rn}.

     u_{rn} is the peak value of the roof displacement due to the nth mode, and is given be

$$u_{rn} = \Gamma_n \phi_{rn} D_n$$

     where $D_n \equiv D(T_n, \xi_n)$ is the ordinate of the deformation response spectrum corresponding to the period T_n and damping ratio ξ_n of the nth mode.

The Modal Pushover Analysis (MPA) Procedure

Linearly Elastic Systems

- The peak modal responses, $\boldsymbol{r_n}$, each determined by one pushover analysis, can be combined according to the modal combination rules (as used in the Response Spectrum Analysis, RSA [SRSS or CQC]) to obtain an estimate of the peak value $\boldsymbol{r}$ of the total response.
- Being equivalent to the standard RSA procedure, the MPA procedure offers no advantage for linearly elastic systems, but this interpretation of RSA permits extension of MPA to inelastic systems.
- Note that $\boldsymbol{r_n}$ determined by pushover analysis can also be interpreted as the peak response of the linearly elastic system to $\boldsymbol{P}_{eff,n}(t)$, the nth-mode component of the effective earthquake forces. This interpretation is valid because, the system responds only in its nth mode when subjected to this excitation.

The Modal Pushover Analysis (MPA) Procedure

Inelastic Systems

- The peak response r_n of the inelastic system to $\boldsymbol{P}_{eff,n}(t)$ is also determined by a pushover analysis, which is now a nonlinear static analysis instead of a linear static analysis, of the structure subjected to lateral forces distributed over the building height according to $\boldsymbol{s}_n^*$ with the forces increased to push the structure up to roof displacement u_{rn}.

$$u_{rn} = \Gamma_n \phi_{rn} D_n$$

  D_n is now the peak deformation of the nth-mode inelastic SDF system (instead of the nth-mode elastic SDF system).

$$\ddot{D}_n(t) + 2\xi_n \omega_n \dot{D}_n(t) + \frac{F_{sn}}{L_n} = -\ddot{u}_g(t)$$

- At u_{rn}, the results of nonlinear static analysis provide an estimate of the peak value r_n of the response quantity $r_n(t)$: floor displacements, story drifts, and other deformation quantities.

The Modal Pushover Analysis (MPA) Procedure

Inelastic Systems

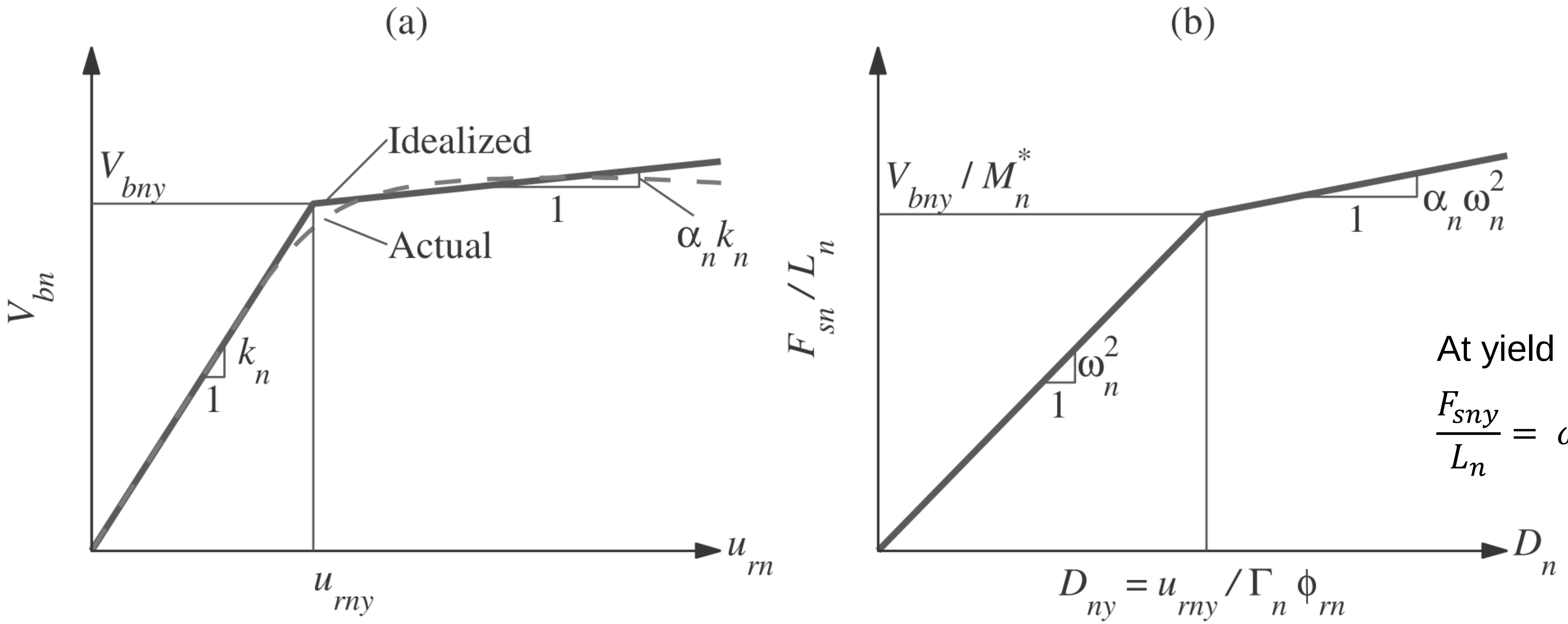
- Nonlinear static analysis using force distribution $\boldsymbol{s}_n^*$ leads to the nth-mode pushover curve, a plot of base shear V_{bn} versus roof displacement u_{rn}.
- From the nth-mode pushover curve is obtained the force–deformation ($F_{sn}/L_n - D_n$) curve for the nth-mode inelastic SDF system, which is required to determine D_n from the following equation.

$$\ddot{D}_n(t) + 2\xi_n\omega_n\dot{D}_n(t) + \frac{F_{sn}}{L_n} = -\ddot{u}_g(t)$$

- The forces and displacements in the two sets of curves are related as follows:

$$D_n = \frac{u_{rn}}{\Gamma_n\phi_{rn}} \qquad \frac{F_{sn}}{L_n} = \frac{V_{bn}}{M_n^*}$$

Where, $M_n^* = L_n\Gamma_n$ is the effective modal mass.



At yield point,

$$\frac{F_{sny}}{L_n} = \omega_n^2 D_{ny}$$

Figure 20.7.2 (a) An *n*th-mode pushover curve and its bilinear idealization; (b) force–deformation relation for the *n*th-mode inelastic SDF system.

UMRHA vs. MPA

- The response value r_n determined by pushover analysis is an estimate of the peak value of the response $r_n(t)$ of the inelastic structure to $\boldsymbol{P}_{eff,n}(t)$, but it is not identical to another estimate determined by UMRHA.

- For elastic systems UMRHA = Modal RHA, and MPA = RSA.

- For inelastic systems the two—UMRHA and MPA—estimates of the peak modal response are both approximate and different from each other; the only exception is the roof displacement because it is deliberately matched in the two analyses.

UMRHA vs. MPA Procedures

- The two estimates differ because the underlying analyses involve different assumptions.

- The UMRHA is based on the approximation contained in $\boldsymbol{u}_n(t) \simeq \boldsymbol{\phi}_n q_n(t)$, which is avoided in MPA because the floor displacements, story drifts, and other deformation quantities are determined by nonlinear static analysis using force distribution $\boldsymbol{s}_n^*$. As a result, the floor displacements of the inelastic system are no longer proportional to the nth-mode shape, in contrast to $\boldsymbol{u}_n(t) \simeq \boldsymbol{\phi}_n q_n(t)$. In this sense, the MPA procedure represents the nonlinear behavior of the structure better than UMRHA.

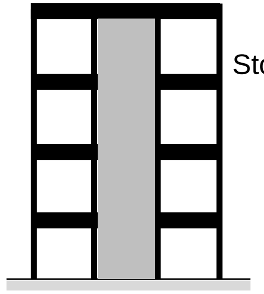
- However, the MPA procedure contains a different source of approximation, which does not exist in UMRHA. The peak modal responses r_n, each determined by one nonlinear static analysis, are combined by a modal combination rule, just as in RSA of linearly elastic systems. This application of modal combination rules to inelastic systems lacks a rigorous theoretical basis, but seems reasonable because the modes are only weakly coupled.

A Modified Response Spectrum Analysis (MRSA) Procedure

Primary Motivation

The Standard RSA Procedure (ASCE 7-10, IBS 2012, EC 8)

Linear Elastic Model



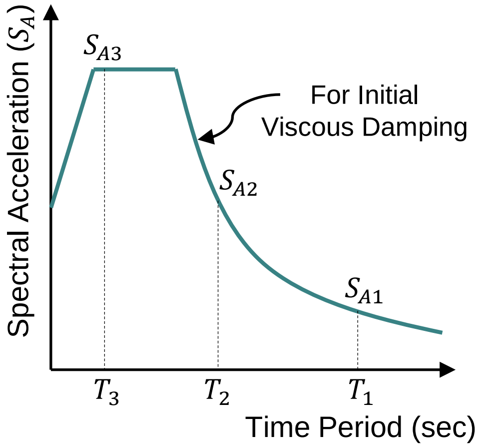
Eigen-value Analysis

$$[\mathbb{K} - \omega^2 \mathbb{M}]\Phi = \mathbb{O}$$

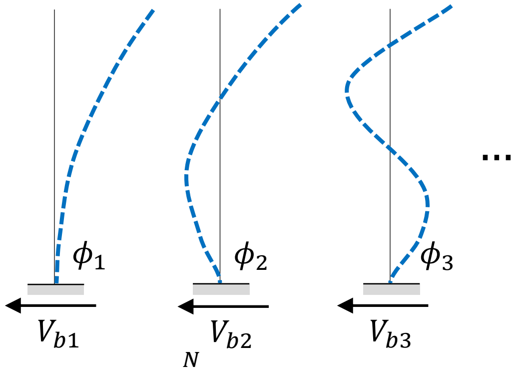
Determine Modal Properties

$$T_i,\ \phi_i,\ \Gamma_i$$

Determine Spectral Acceleration for each Significant Mode



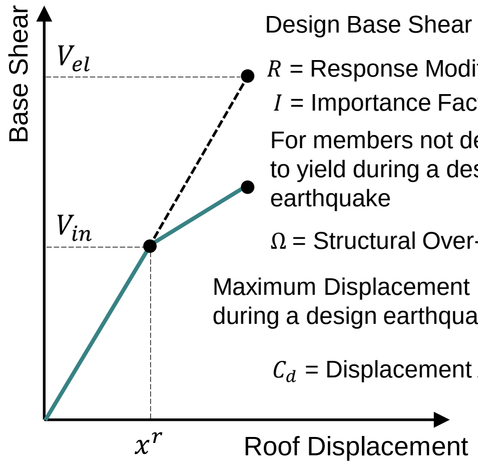
Determine Equivalent Elastic Forces



$$V_{bi} = \sum_{n=1}^{N} \Gamma_i \,.\, m_n \,.\, \phi_{i,n} \,.\, S_{Ai}$$

$$V_{el} = \sqrt{(V_{b1})^2 + (V_{b2})^2 + (V_{b3})^2 + \ \ldots}$$

Determine Design Demands



Design Base Shear $V_D = V_{in} = \dfrac{V_{el}}{R/I}$

R = Response Modification Factor

I = Importance Factor

For members not desired to yield during a design earthquake $V_D = \Omega \dfrac{V_{el}}{R}$

Ω = Structural Over-strength Factor

Maximum Displacement during a design earthquake $x^r_{max} = \dfrac{C_d x^r}{I}$

C_d = Displacement Amplification Factor

Criticisms on the RSA Procedure

(a) $$\text{Inelastic Demands} = \frac{\text{Elastic Demands}}{\text{Some Factor}}$$

(b) $$\text{Inelastic Demands of Each Mode} = \frac{\text{Elastic Demands of That Mode}}{\text{Same Factor}}$$

(c) Statistical Combination of Modal Demands

(d) Static Analysis Procedure (or Pseudo-dynamic, to say the least)

Newmark & Hall (1982)
Eibl & Keintzel (1988)
Bertero (1986)
Miranda & Bertero (1994)
ATC 19 (1995)
ATC-34 (1995)
Cuesta & Aschheim (2001)
Priestley & Amaris (2002)
Foutch & Wilcoski (2005)
Sullivan, Priestley & Calvi (2008)
Maniatakis et al. (2013)
.
.
.

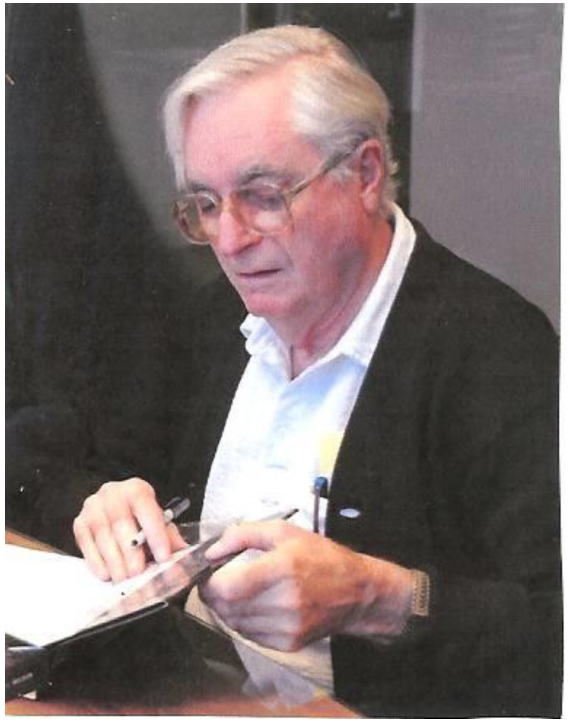
Criticisms on the RSA Procedure

"Ray Clough and I regret we created the approximate response spectrum method for seismic analysis of structures in 1962."

"… allowed engineers to produce meaningless positive numbers of little or no value."

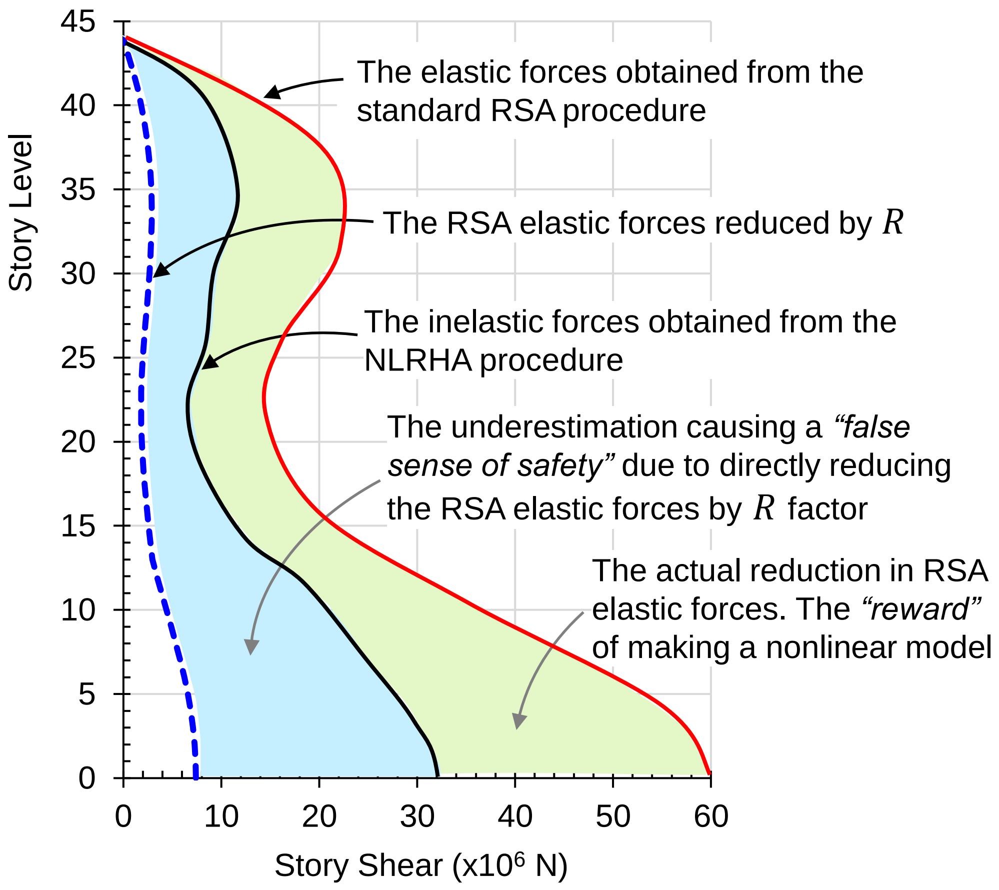
"Do not be called a Neanderthal man."

CASE CLOSED
The use of the Response Spectrum Method in Earthquake Engineering must be terminated.
It is not a dynamic analysis method – The results are not a function of time.



– *Edward L. Wilson*
Professor Emeritus (CEE, UC Berkeley)

The Problem with R Factor



The Basic Concept of the Modified Response Spectrum Analysis (MRSA)

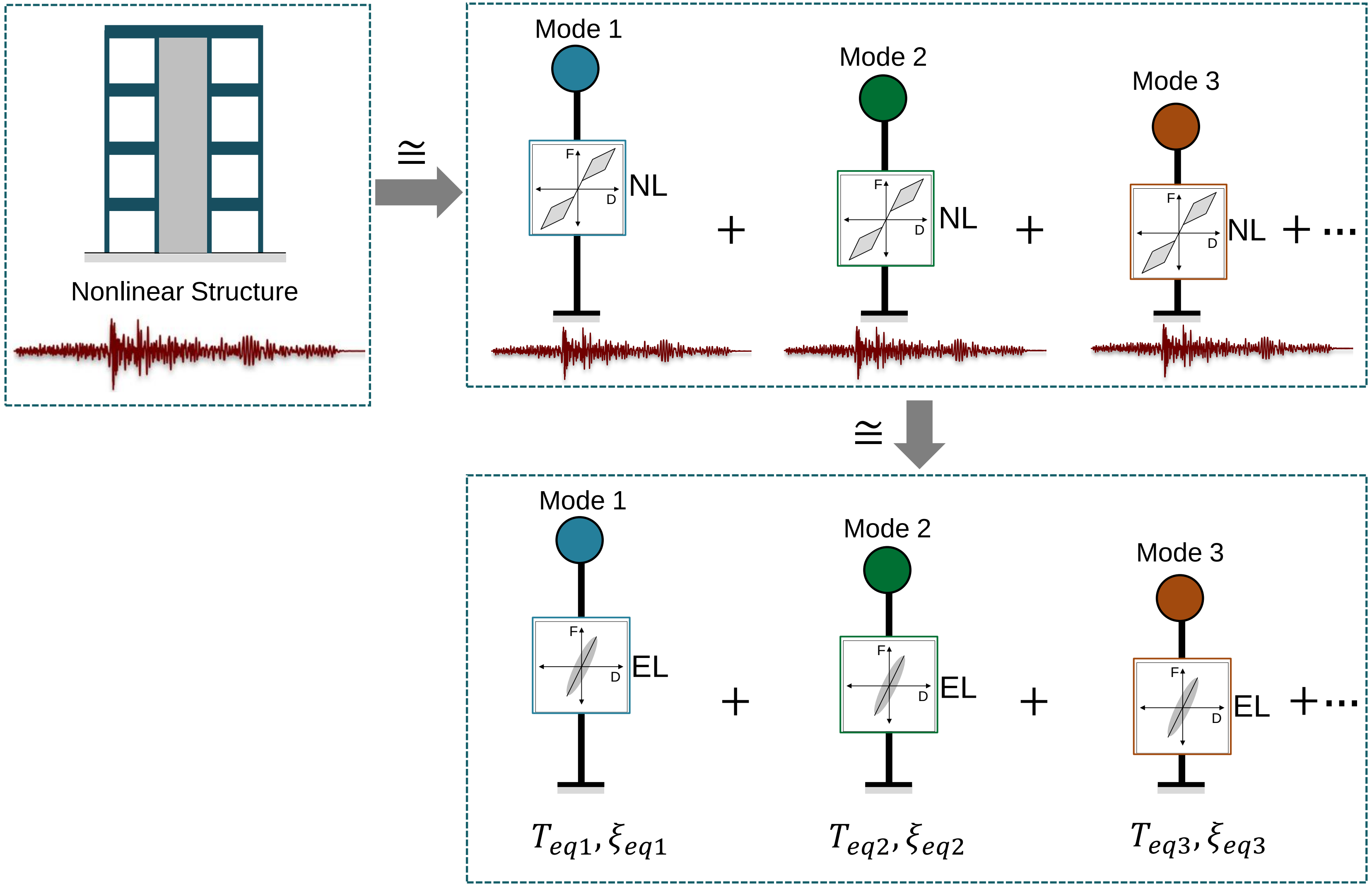
NLRHA

↓

UMRHA

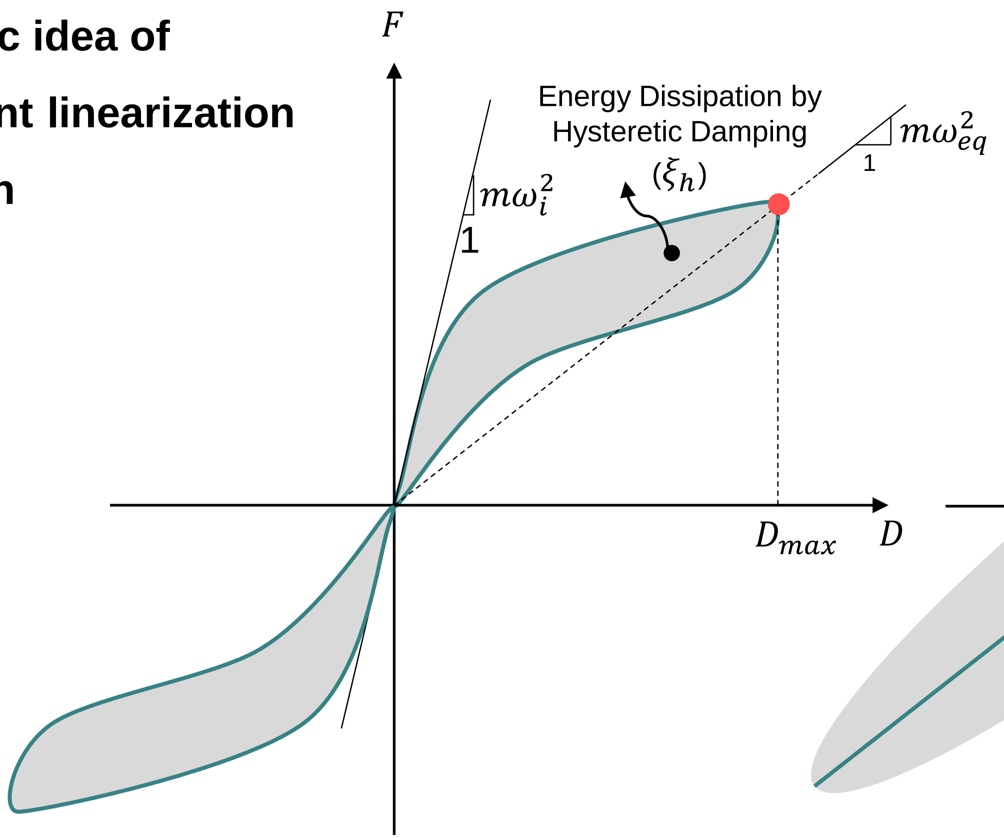
↓

MRSA

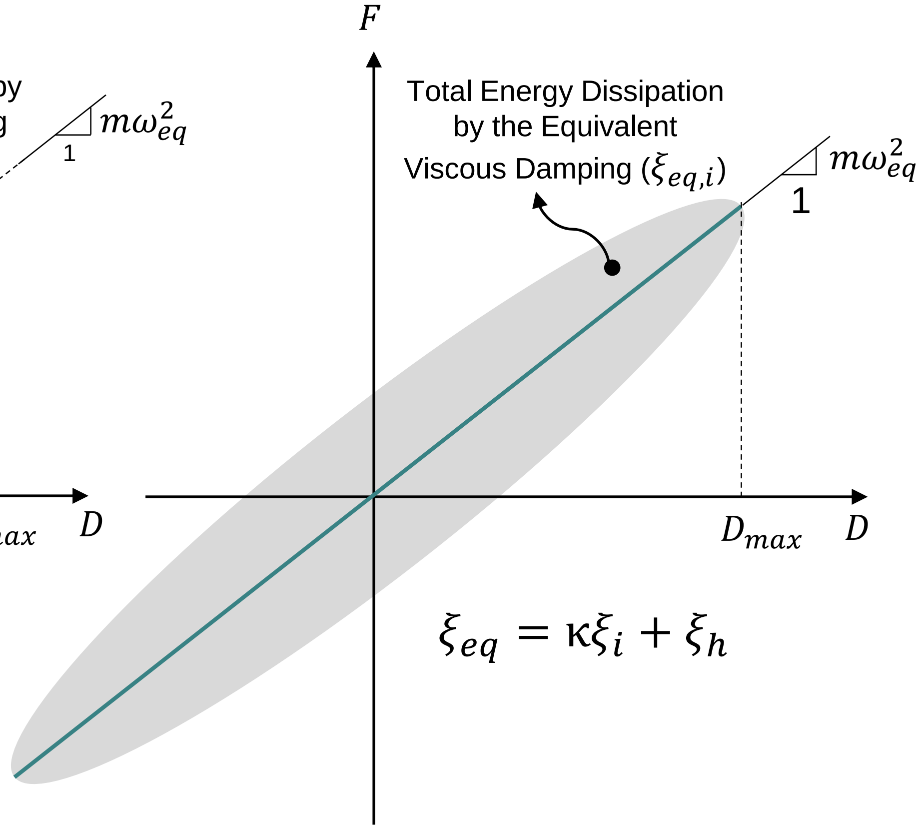


Conversion of a Nonlinear SDF System into an "Equivalent Linear" SDF System

The basic idea of equivalent linearization approach



A nonlinear SDF system with initial circular natural frequency ω_i, and with initial inherent viscous damping ξ_i



$$\xi_{eq} = \kappa\xi_i + \xi_h$$

An Equivalent Linear System with Elongated Period and Additional Damping

$$T_{eq} = \frac{2\pi}{\omega_{eq}}$$

ξ_{eq} and T_{eq} are the "equivalent linear properties"

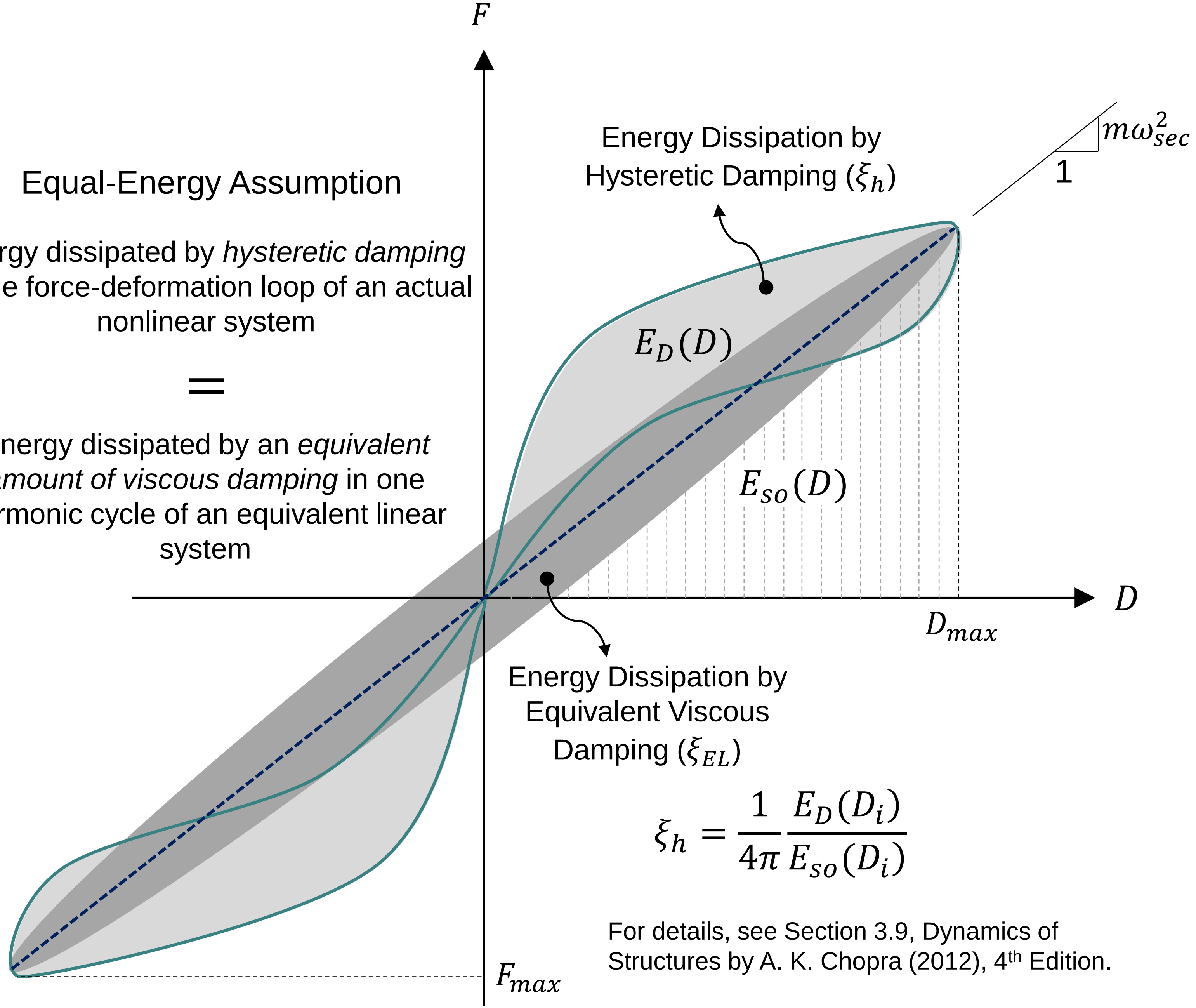
Determination of Hysteretic Damping – Equal-energy Dissipation Rule

Equal-Energy Assumption

Energy dissipated by *hysteretic damping* in one force-deformation loop of an actual nonlinear system

=

Energy dissipated by an *equivalent amount of viscous damping* in one harmonic cycle of an equivalent linear system

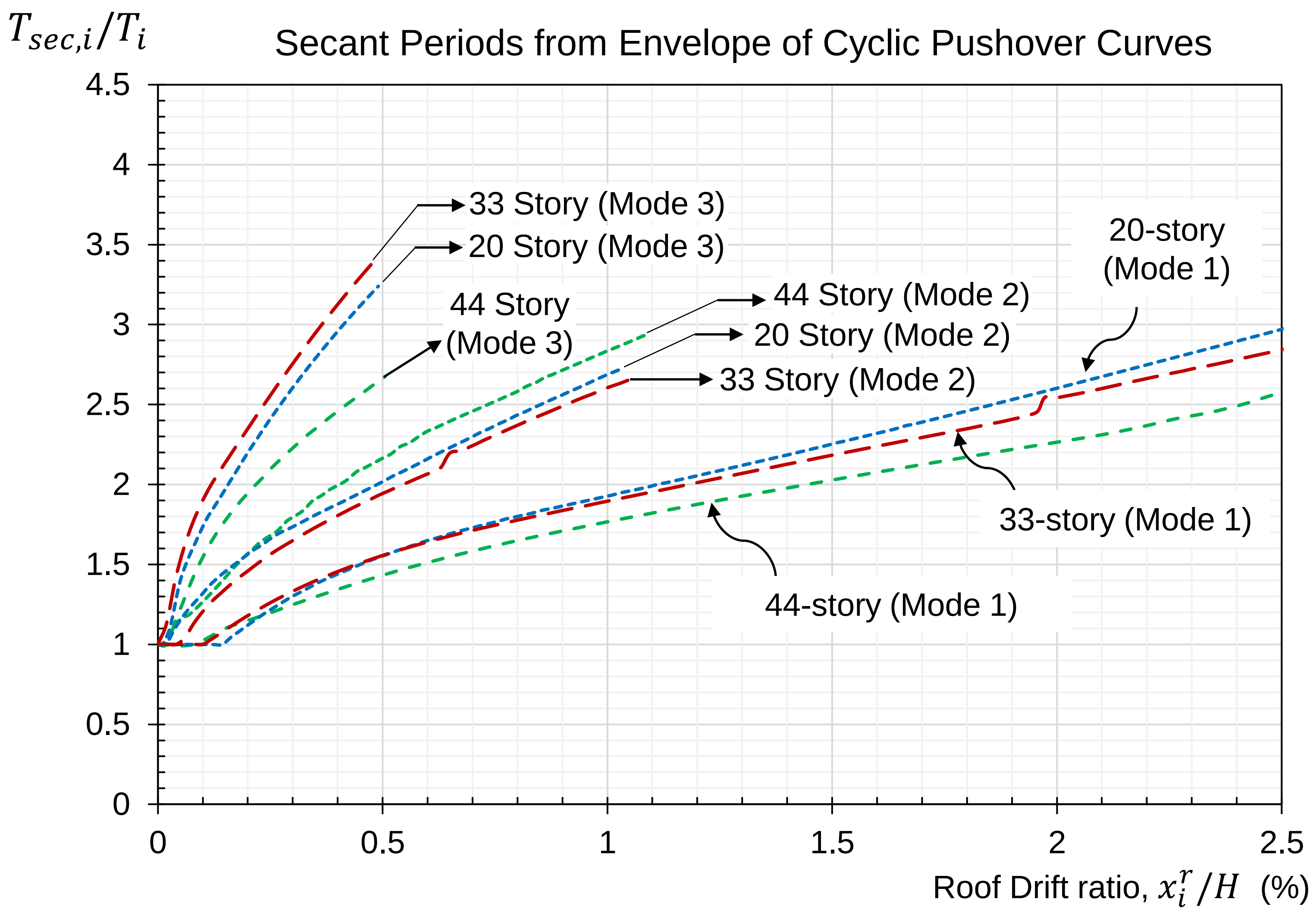


$$\xi_h = \frac{1}{4\pi}\frac{E_D(D_i)}{E_{so}(D_i)}$$

For details, see Section 3.9, Dynamics of Structures by A. K. Chopra (2012), 4th Edition.

“*Equivalent Linear*”

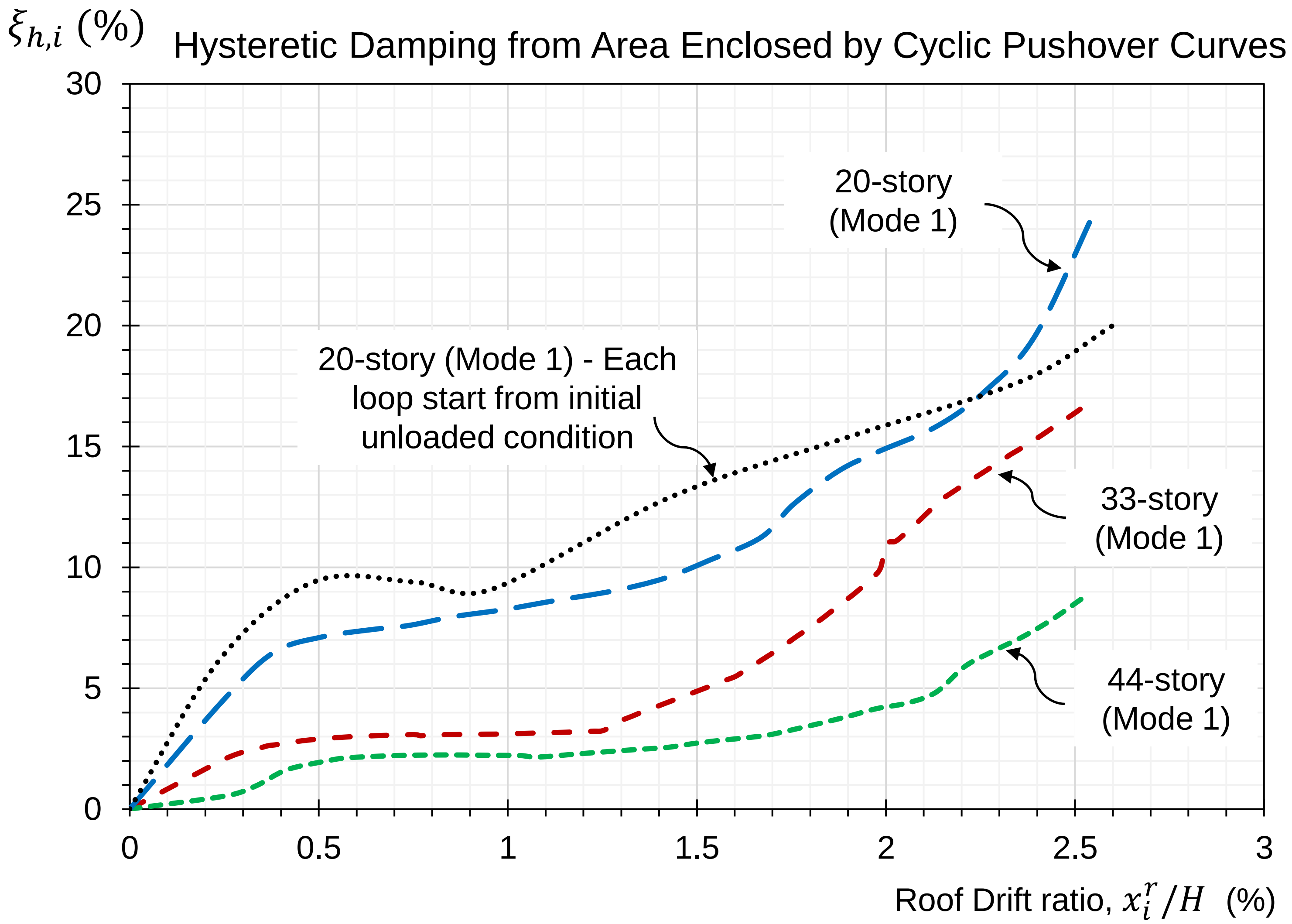
Properties



$T_{sec,i}/T_i$ vs. Roof Drift Ratio (x_i^r/H)

“Equivalent Linear”

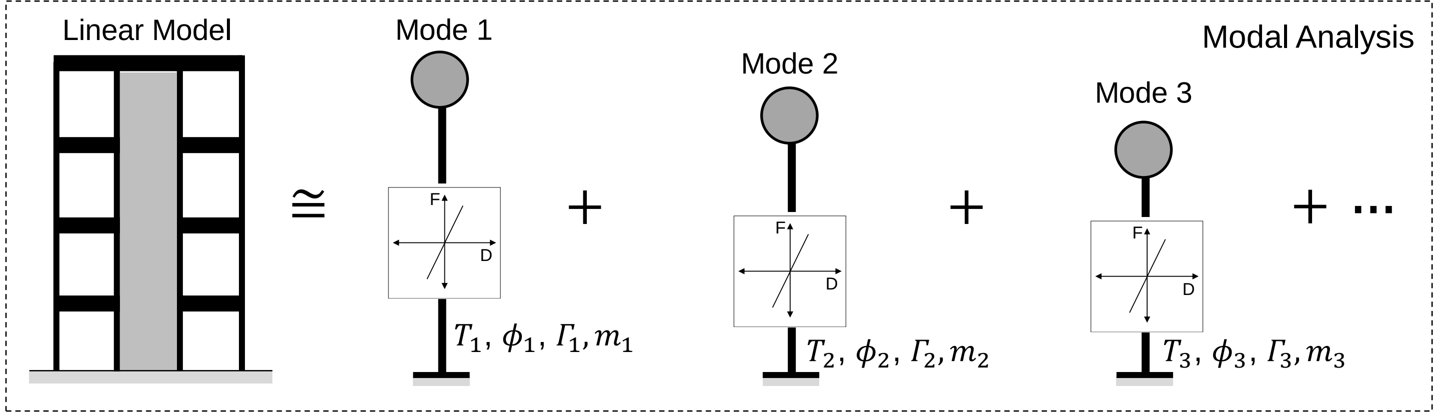
Properties

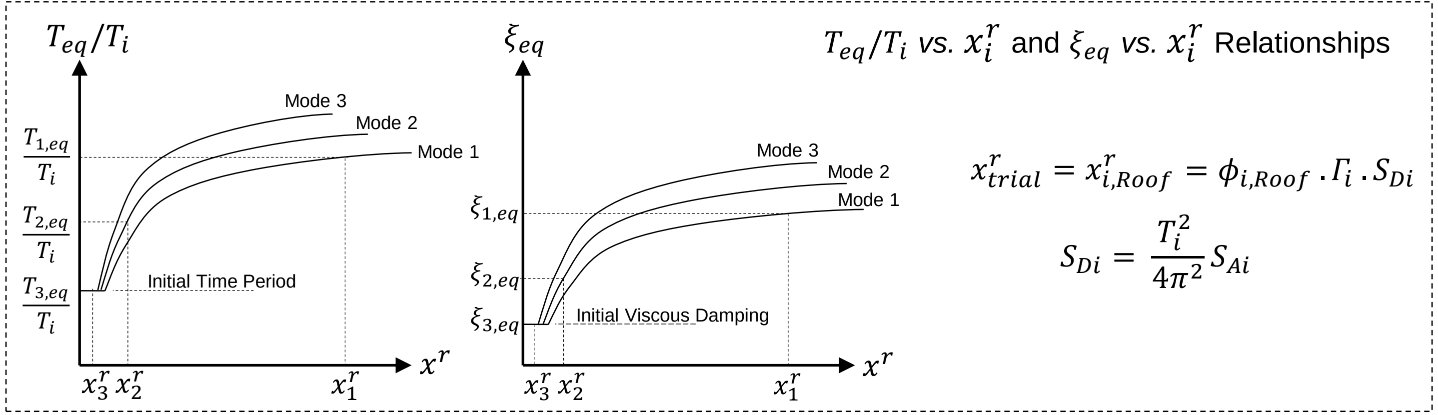


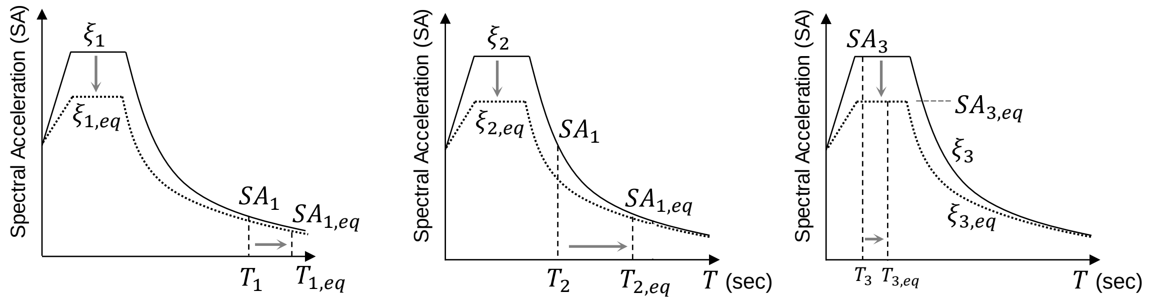
(b) $\xi_{h,i}$ *vs. Roof Drift Ratio* (x_i^r/H)

The Modified Response Spectrum Analysis (MRSA)

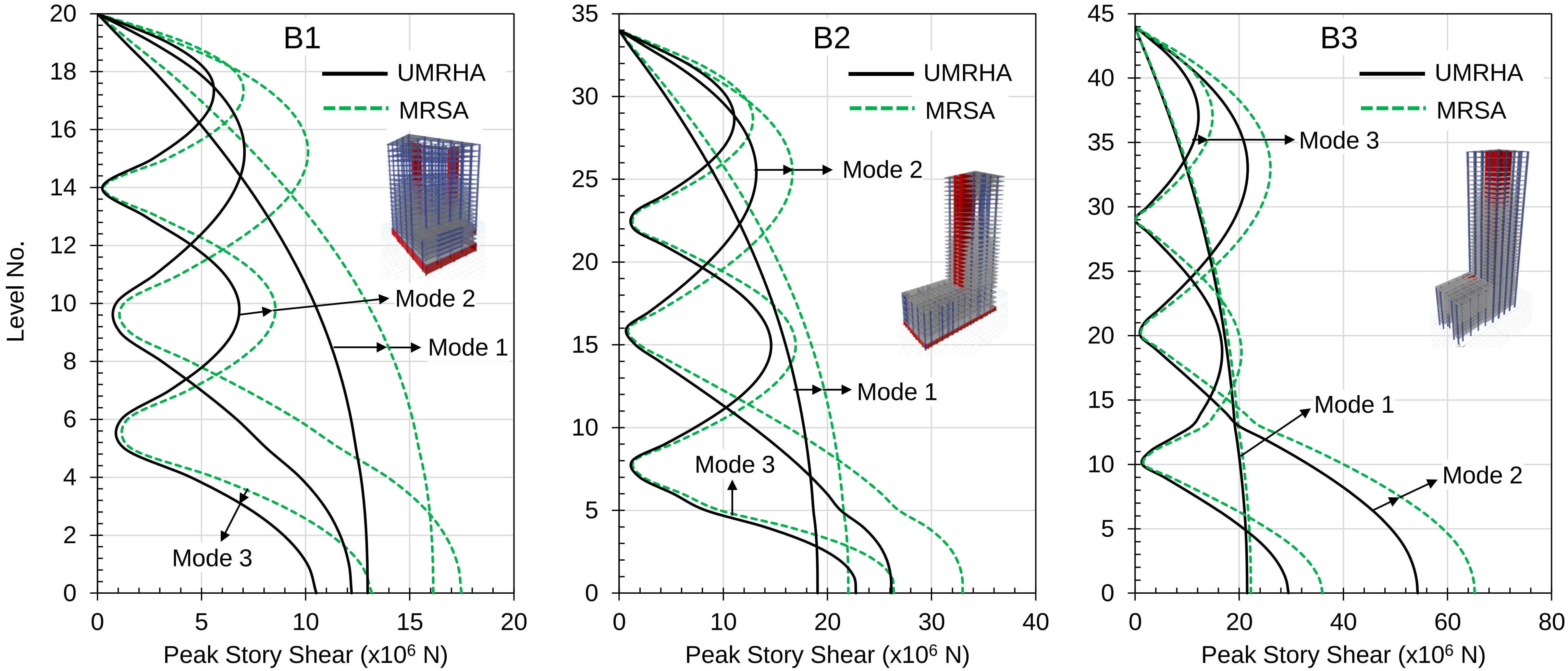
What is *“Modified”*?







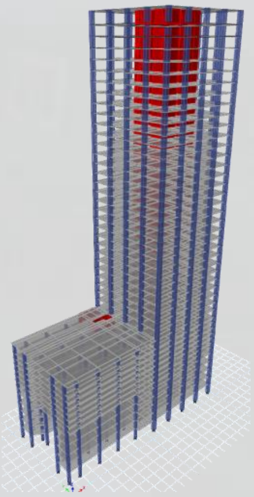
The MRSA Procedure – Individual Modal Demands – Ground Motion Set 4

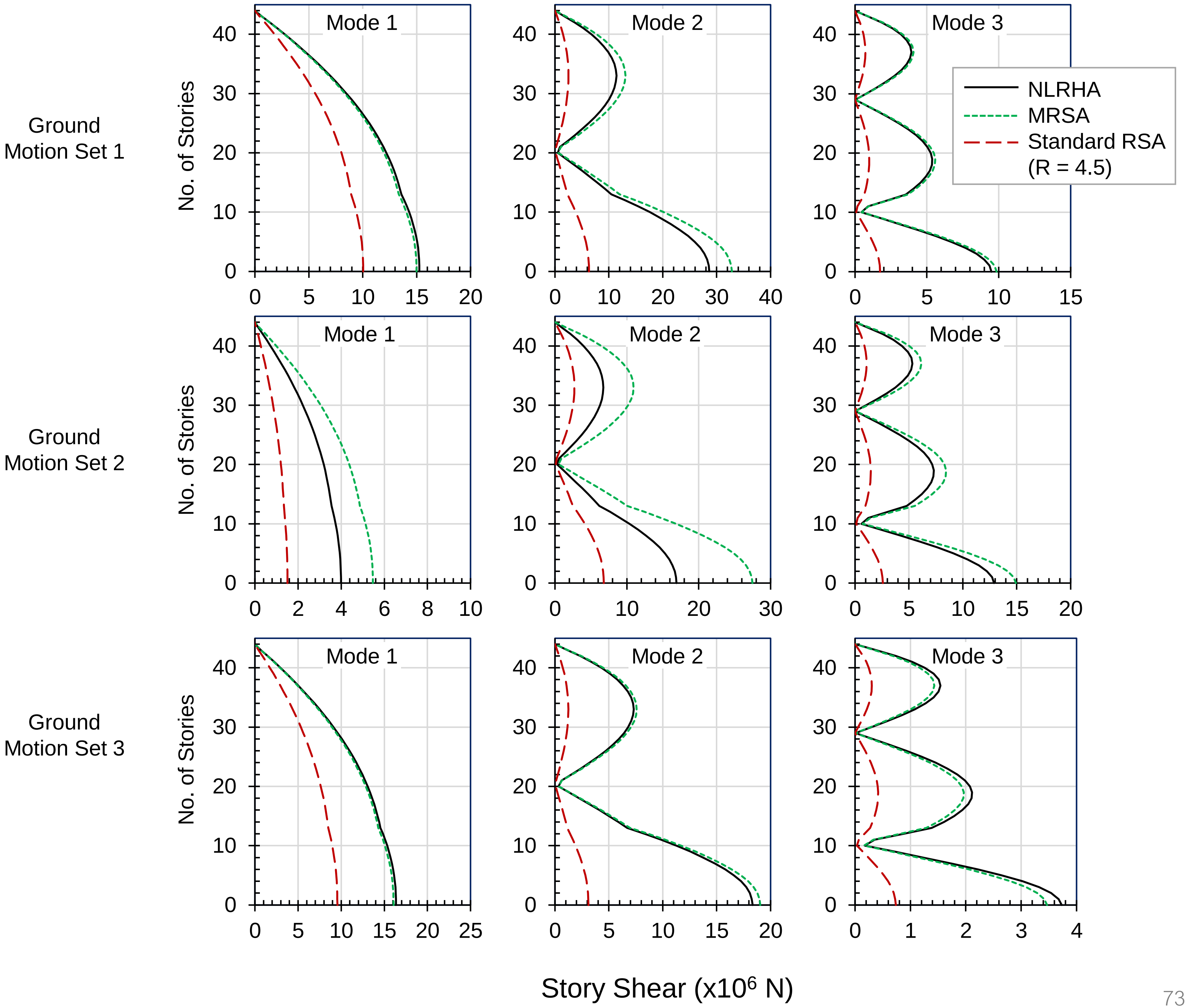


Performance of the MRSA Procedure – Individual Modal Demands

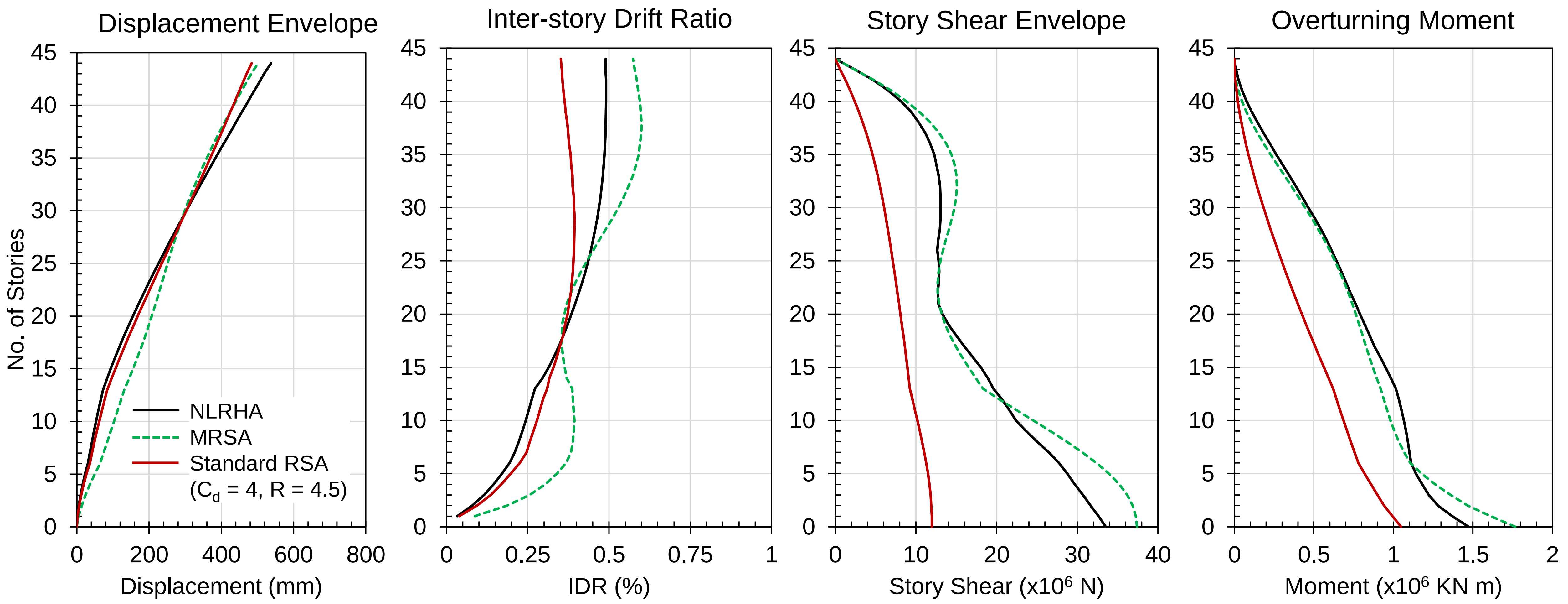
Ground Motion Sets 1, 2 and 3

B3

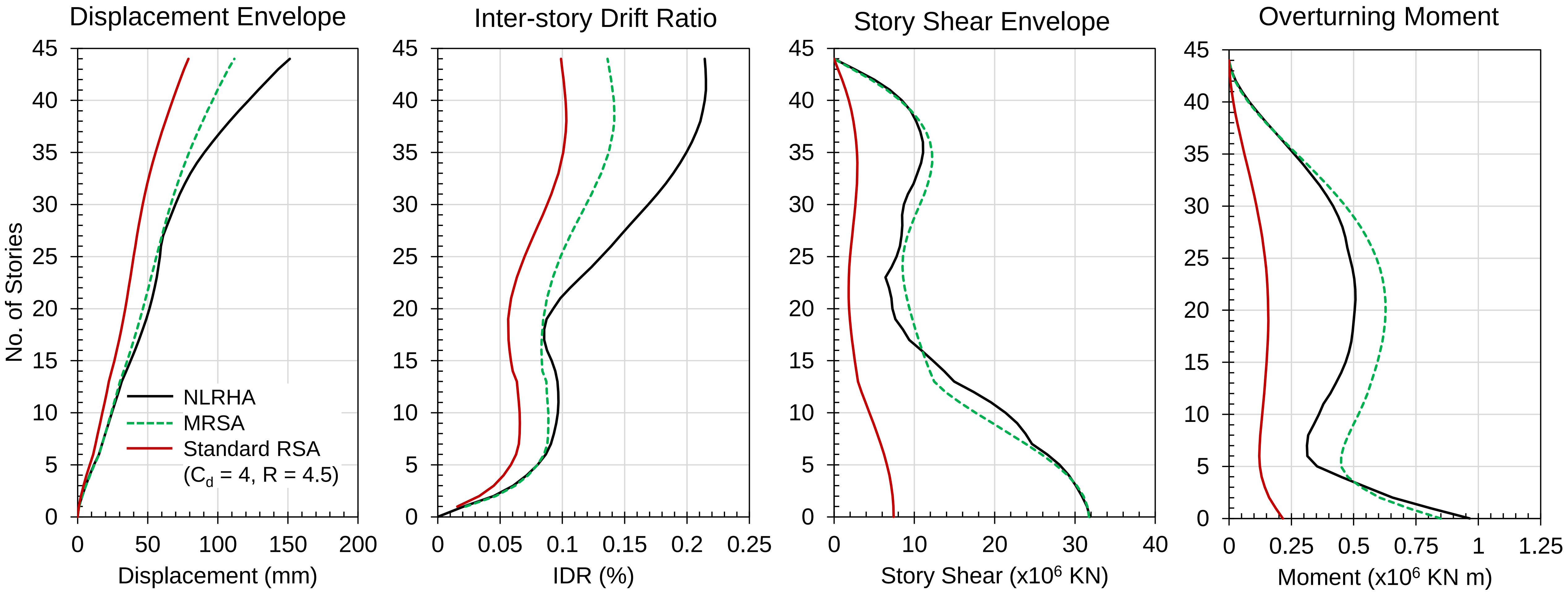




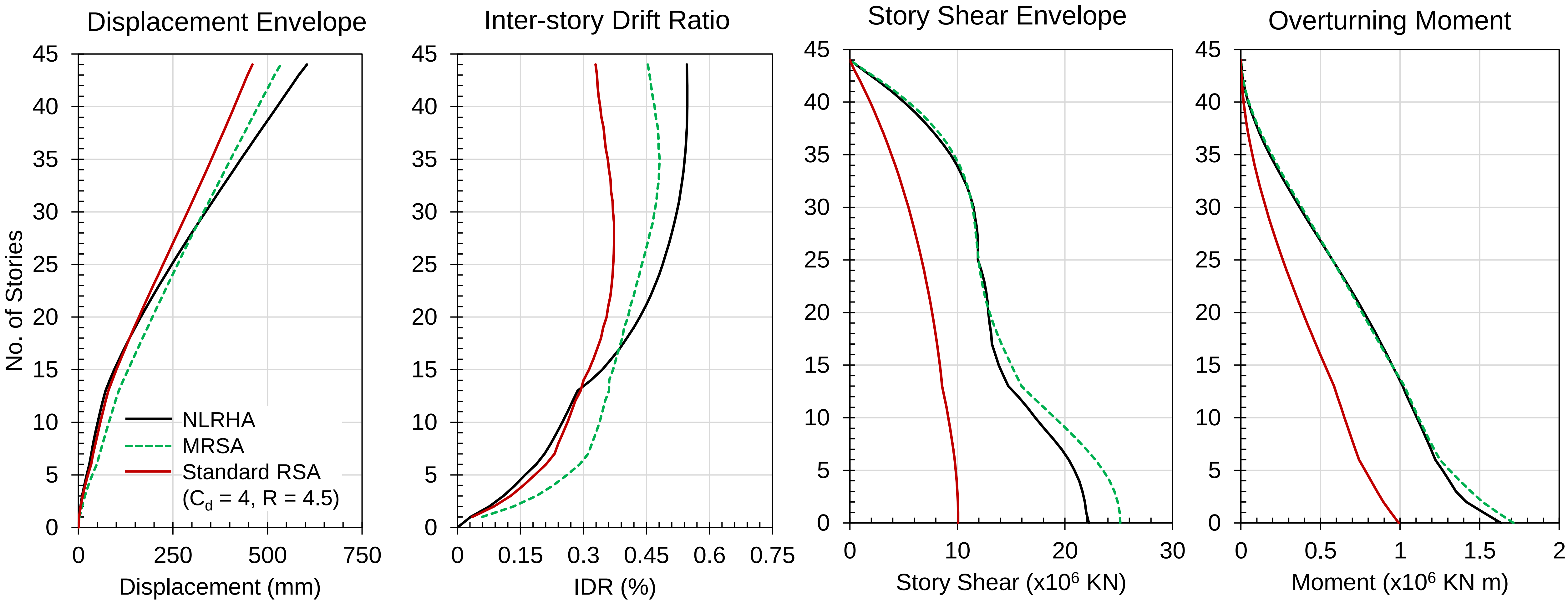
Overall Performance of the MRSA Procedure – B3 – Ground Motion Set 1



Overall Performance of the MRSA Procedure – B3 – Ground Motion Set 2

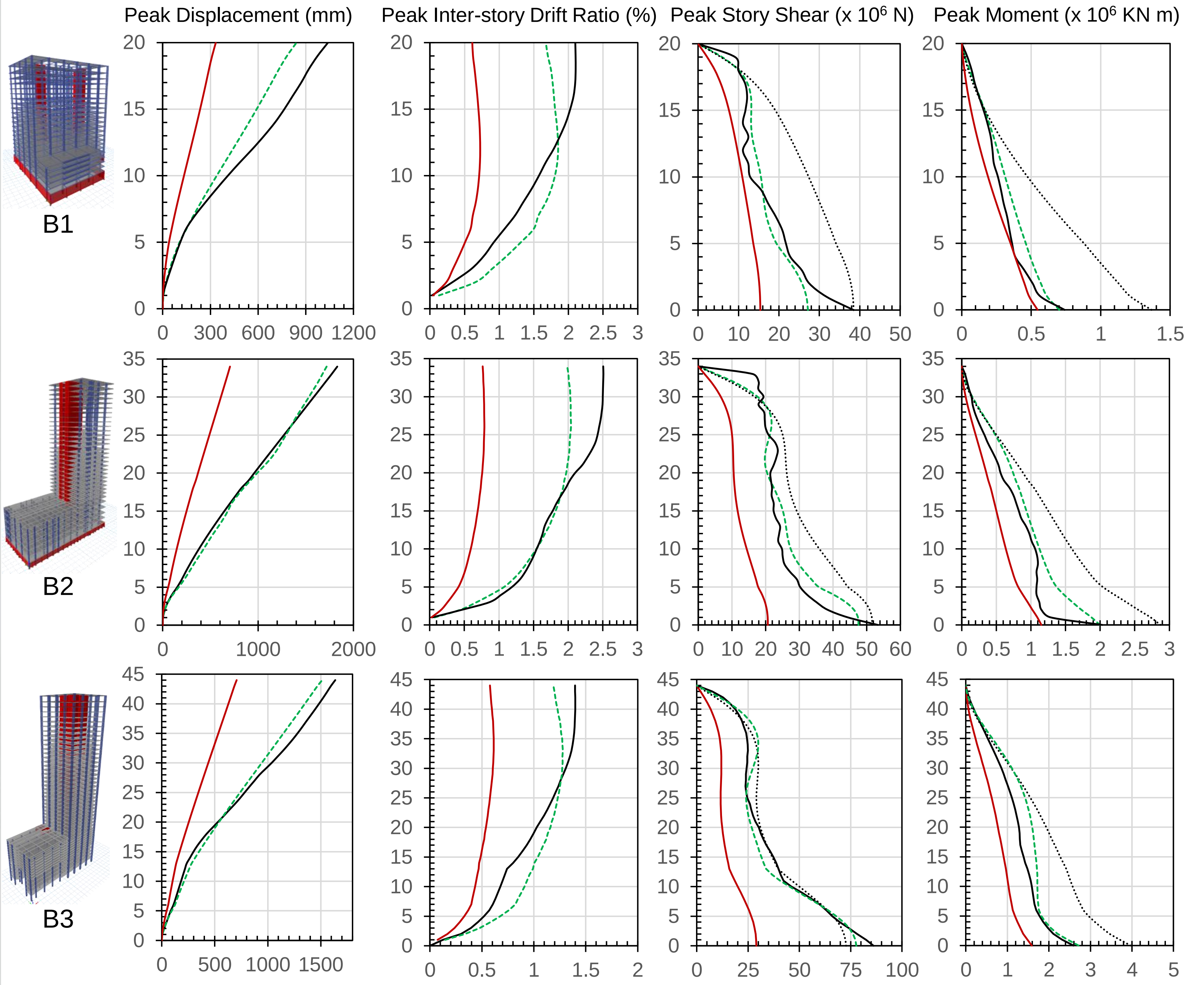


Overall Performance of the MRSA Procedure – B3 – Ground Motion Set 3



Overall Performance of MRSA

Ground Motion Set 4

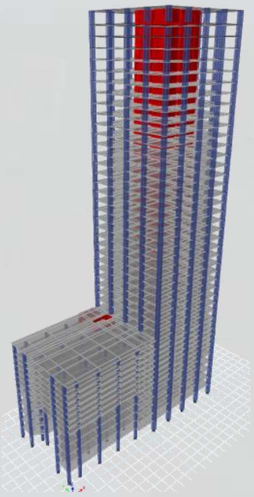


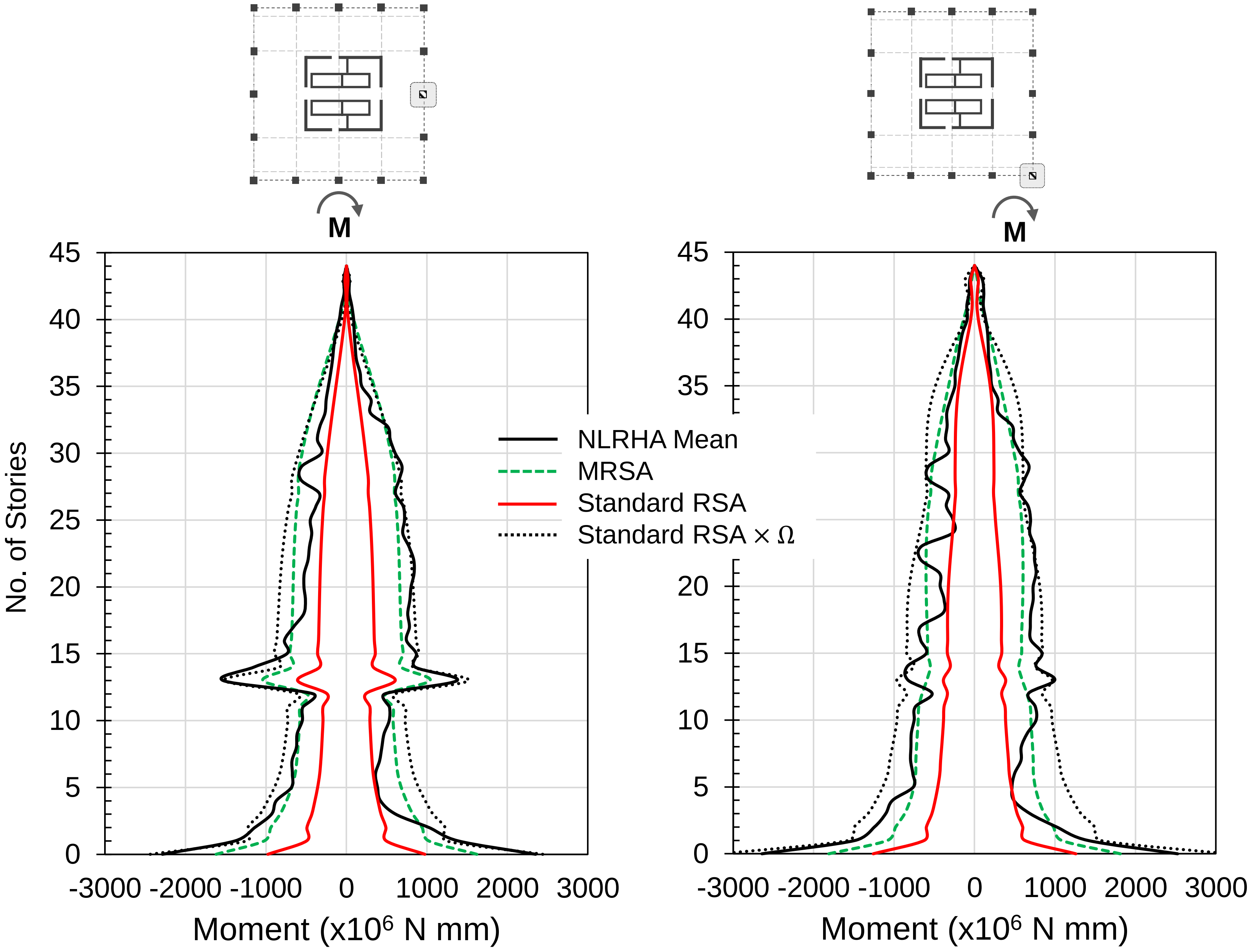
- NLRHA
- MRSA
- Standard RSA (C_d = 4, R = 4.5)
- Standard RSA × Ω

Bending Moments in Columns

Ground Motion Set 4

B3

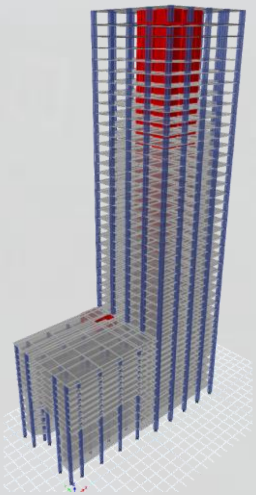


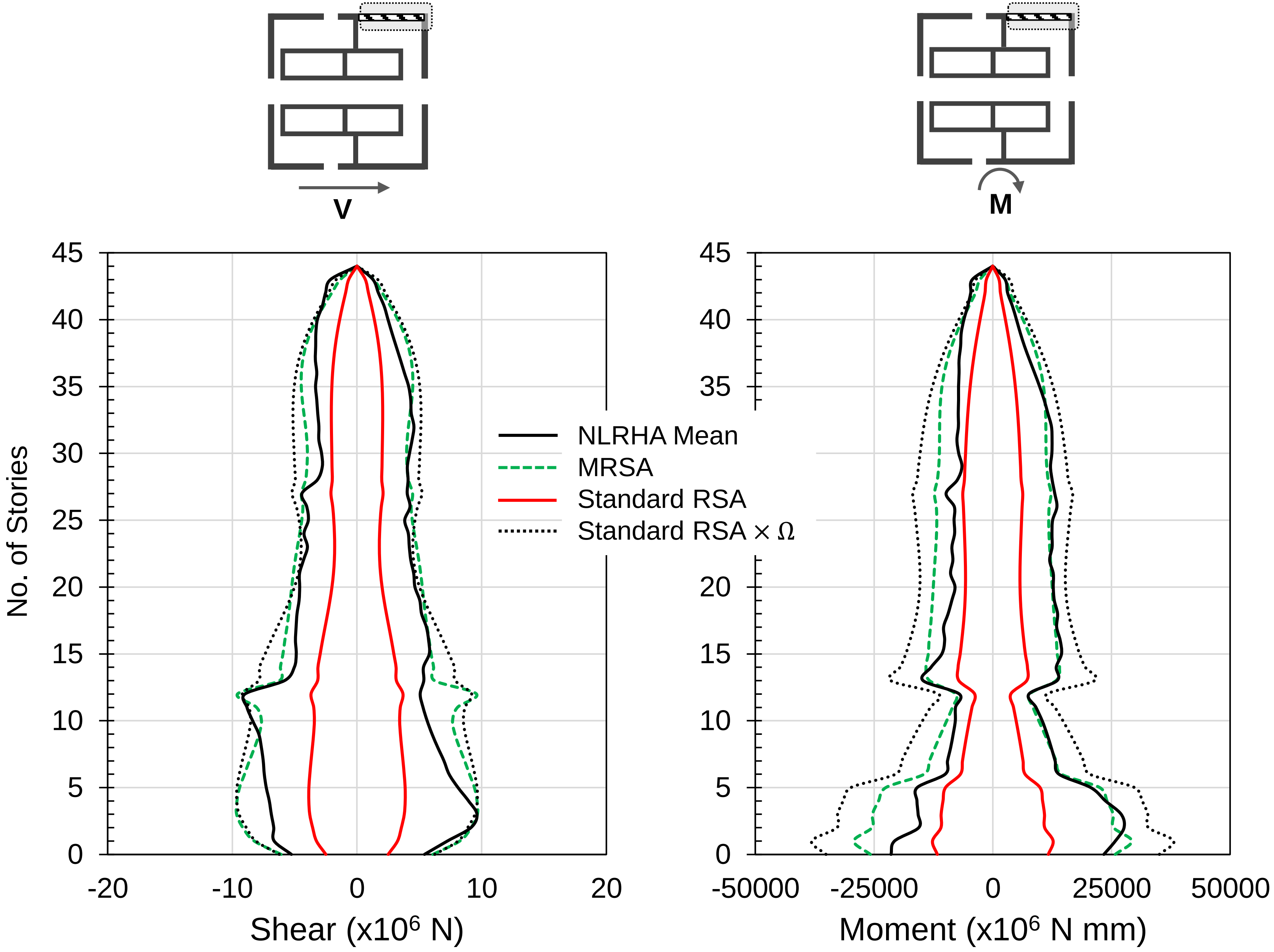


Shear Forces and Bending Moments in Shear Walls

Ground Motion Set 4

B3





Summary and Conclusions

- The proposed MRSA procedure works
- Requires significantly less computational time and effort compared to the detailed NLRHA procedure
- Simple, conceptually superior, provides mode-by-mode response, clear insight
- Doesn’t require nonlinear modeling
- Can be considered as a convenient analysis and design option

Thank you for your attention