# Fast and Efficient Design for Stability

Larry Griffis
Rafael Sabelli

NASCC: THE STEEL CONFERENCE

1

## Session topics

- Stability-design overview – AISC Methods
- Story Drift and Drift Limits – *wind and seismic*
- Indirect Analysis Method **– *NEW!***
- Background of the AISC $R_M$ factor
- Second-order drift methods
- Example

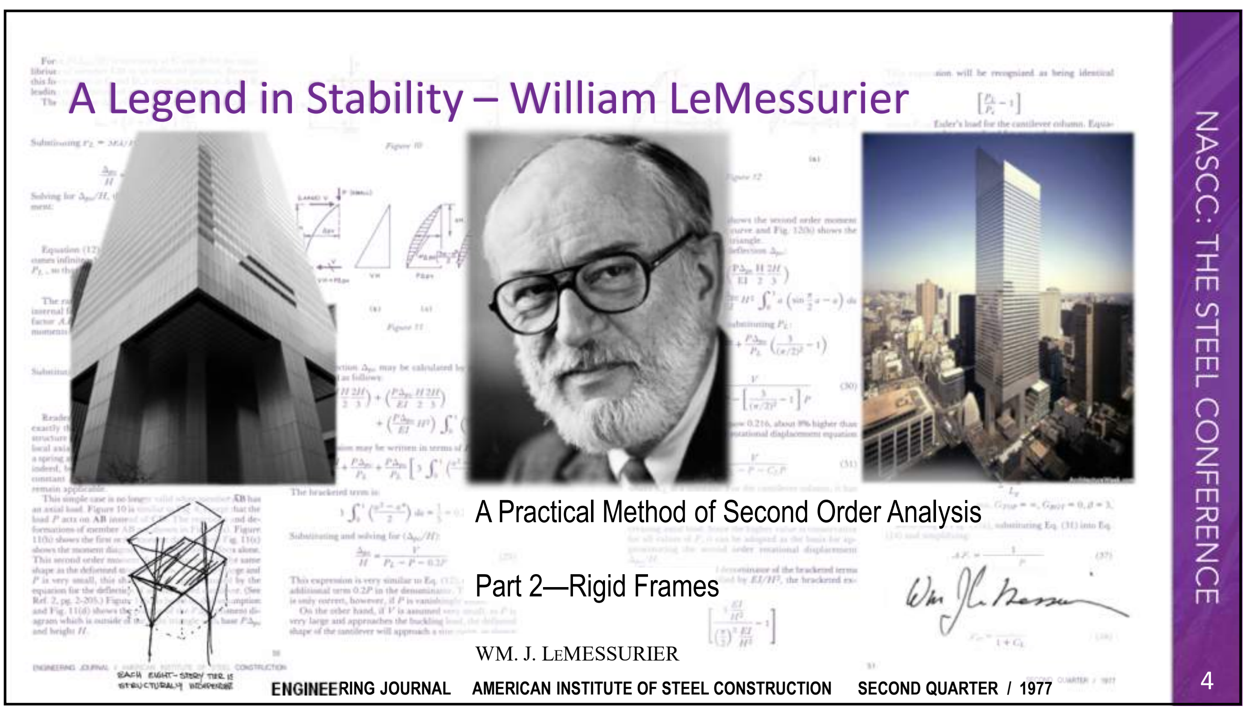
NASCC: THE STEEL CONFERENCE

2

2

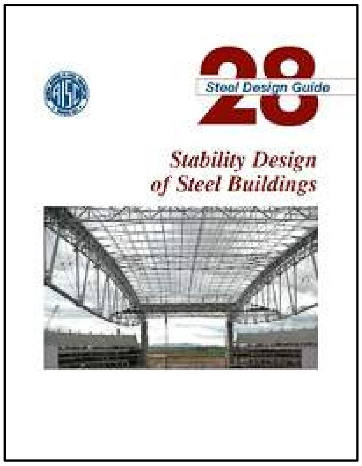

Stability-design
Summary
NASCC:
THE STEEL CONFERENCE

3


A Legend in Stability – William LeMessurier
A Practical Method of Second Order Analysis
Part 2—Rigid Frames
WM. J. LEMESSURIER
ENGINEERING JOURNAL
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
SECOND QUARTER / 1977
NASCC: THE STEEL CONFERENCE
4

4

## AISC DESIGN GUIDE 28




Authors:

Don White

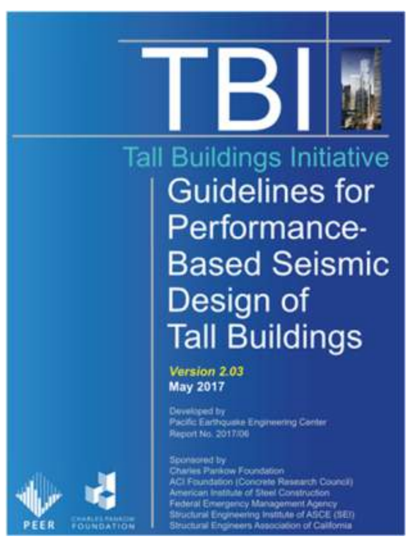
Larry Griffis

NASCC: THE STEEL CONFERENCE

5

5

## Performance Based Seismic Design Tall Buildings - PEER




NASCC: THE STEEL CONFERENCE

6

6

## Stability and Instability – SSRC Guide

Stability
(SSRC Guide – Galambos 1998)

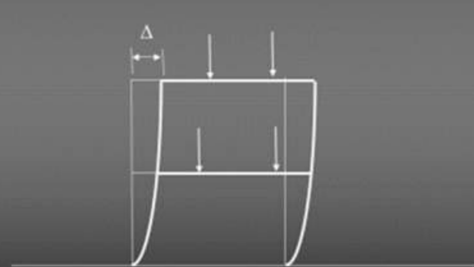
- *Stability* – the capacity of a compression member, element or frame to remain in position and support load, even if forced slightly out of line or position by an added lateral force
- *Instability* – a condition reached during buckling under increasing load in a compression member, element or frame at which the capacity for resistance to added load is exhausted and continued deformation results in a decrease in load carrying capacity

NASCC: THE STEEL CONFERENCE 7

7

## What is stability?

*"Equilibrium in a deformed position under applied load"*




NASCC: THE STEEL CONFERENCE 8

8

## Design for stability

Design considerations

- Strength
  - Adequate to resist loads
- Stiffness
  - Prevent excessive displacement
- Stability
  - Not a separate consideration
  - Increases strength demand
  - Decreases stiffness





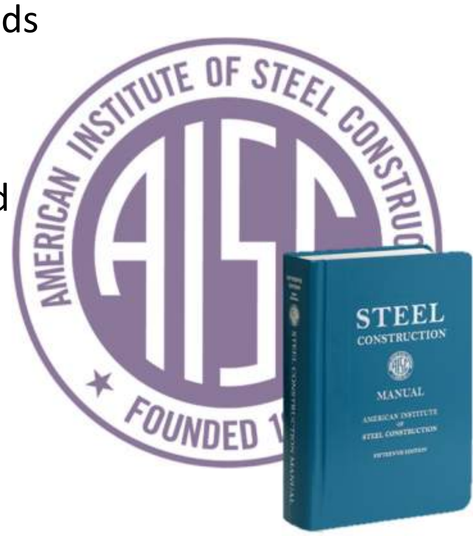


## Design for stability

General requirements for all methods

AISC-defined methods

- FOM: First-order method
- ELM: Effective length method
- DM: Direct analysis method



## Stability Requirements
## *"The Big 5" (Chapter C)*

- *All deformations considered*
  - *flexural, shear, axial, panel zone, etc.*
- *Second order analysis*
  - *P-Δ, P-δ effects*
- *Geometric imperfections*
  - Out-of-straightness
  - Out-of-plumbness
- *Member stiffness reduction due to residual stresses*
  - EI reduction due to premature yielding
- *Uncertainty in stiffness strength*

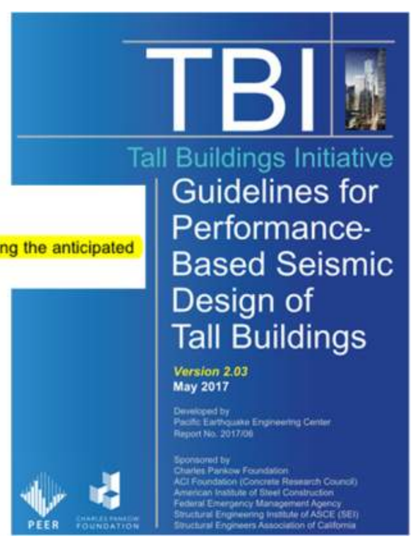
NASCC: THE STEEL CONFERENCE 11

11

## Performance Based Design
## The Analysis Model

**4.6 STRUCTURAL MODELING PARAMETERS**

Develop the structural analysis model based on expected properties considering the anticipated level of response and damage.




NASCC: THE STEEL CONFERENCE 12

12

## Consider all deformations

Axial

Flexural

Shear

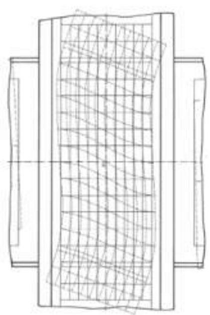
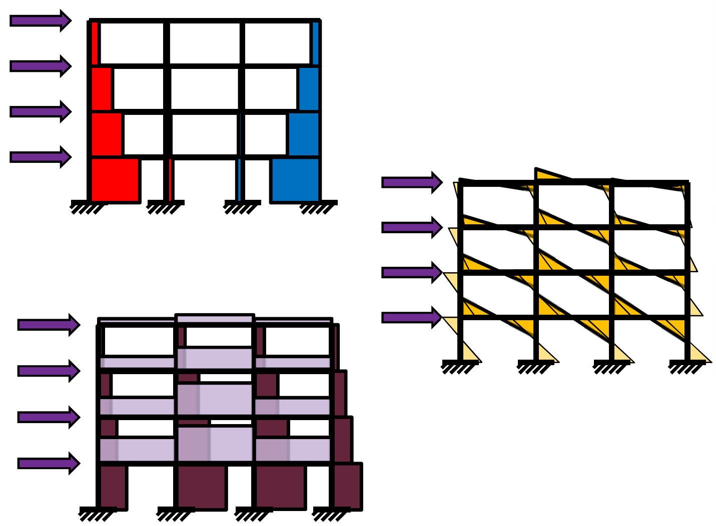


Figure 3-12. Panel zone inelastic deformation (exaggerated)

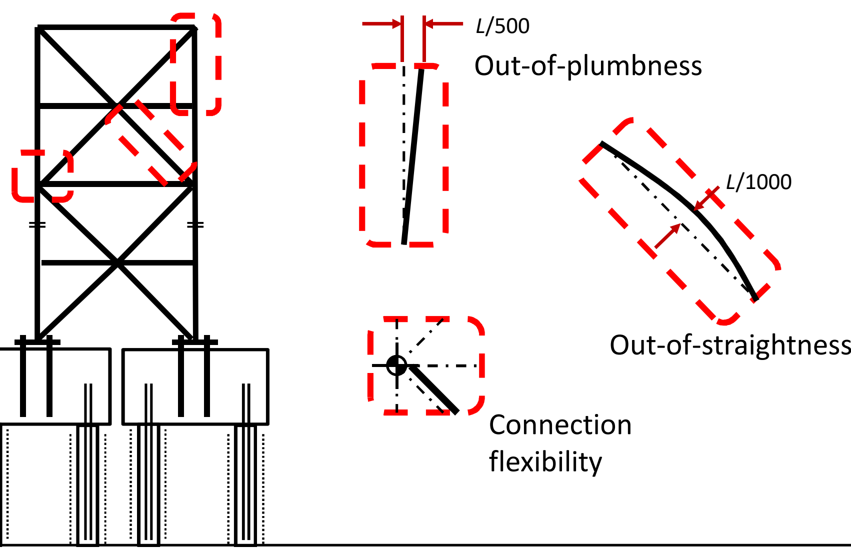


NASCC: THE STEEL CONFERENCE

13

13

## Geometric imperfections




NASCC: THE STEEL CONFERENCE

14

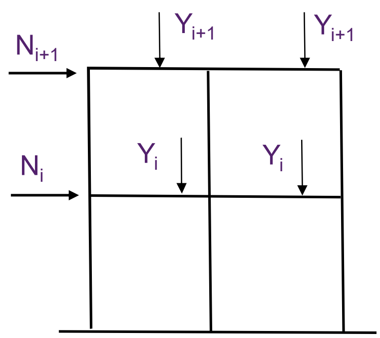
14

## What are *"notional loads"*?

*"Lateral loads applied at each story expressed as a fraction (0.002) of the gravity load at a story"*

$N_i = \sum 0.002\, Y_i$

*Now part of DM and ELM!*

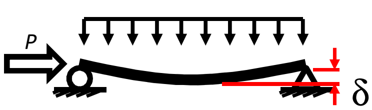



NASCC: THE STEEL CONFERENCE 15

15

## *P*δ (member) vs *P*Δ (system)

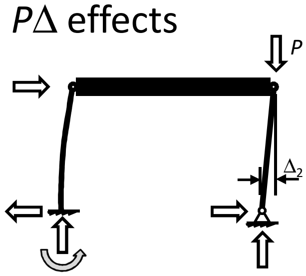
### *P*δ effects

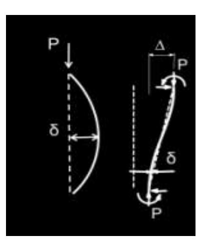



*P*-Δ = Additional moment due to the member axial force acting thru the relative transverse displacement of the member ends

*P*-δ = additional moment due to the member axial force acting thru the transverse displacement of the cross-section relative to a chord between the member ends

### *P*Δ effects




Note: δ influences Δ

NASCC: THE STEEL CONFERENCE 16

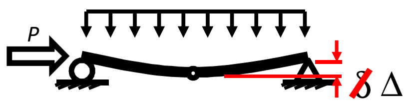
16

## *P*δ modeling

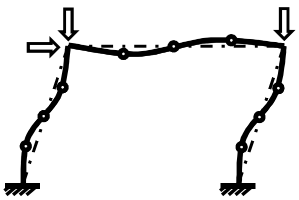
Software typically addresses *P*Δ effects at nodes

*P*δ effects can be modeled

- Introduce nodes: δ → Δ



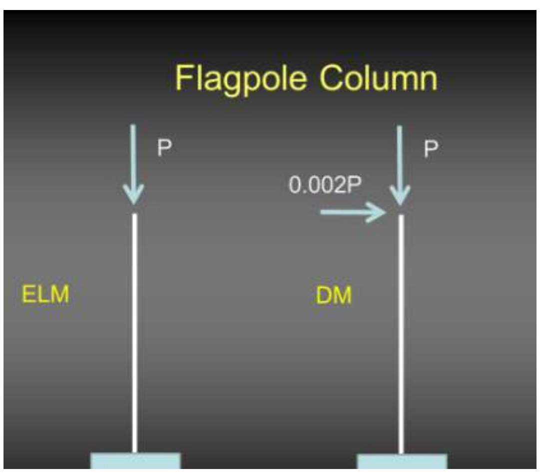

- 4 segments/member



NASCC: THE STEEL CONFERENCE 17

17

## Notional load for the Direct Analysis Method




NASCC: THE STEEL CONFERENCE 18

18

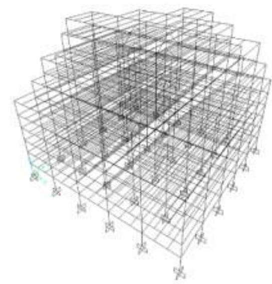
## Big 5 considerations

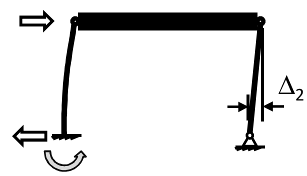
All deformations

- Typically addressed in modeling

Second-order effects

- Second-order analysis






NASCC: THE STEEL CONFERENCE

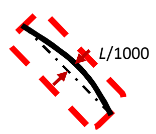
19

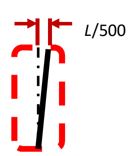
19

## 5 considerations

Geometric imperfections

- Member
  - Reduced design strength
  - Modeling of imperfections
- System
  - Notional load
  - Modeling of imperfections




NASCC: THE STEEL CONFERENCE

20

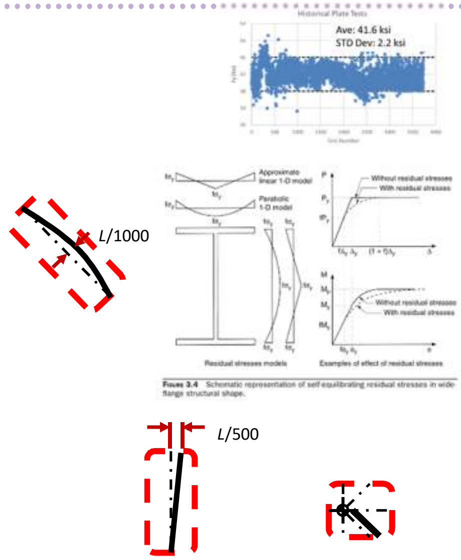
20

## 5 considerations

Stiffness reduction due to inelasticity & uncertainty in strength and stiffness

- Member
  - Reduced strength

    $\phi$
  - Modeling



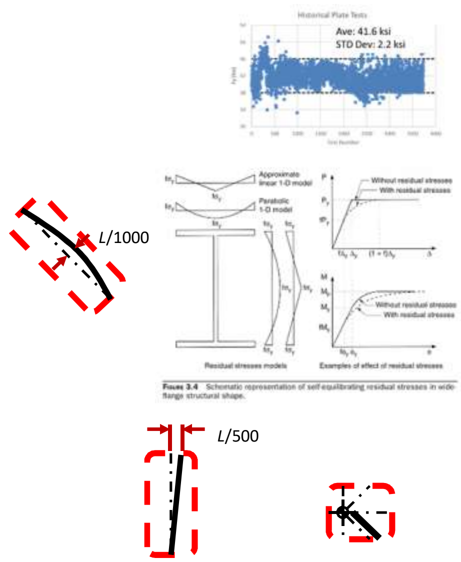

NASCC: THE STEEL CONFERENCE 21

21

## 5 considerations

Stiffness reduction due to inelasticity & uncertainty in strength and stiffness

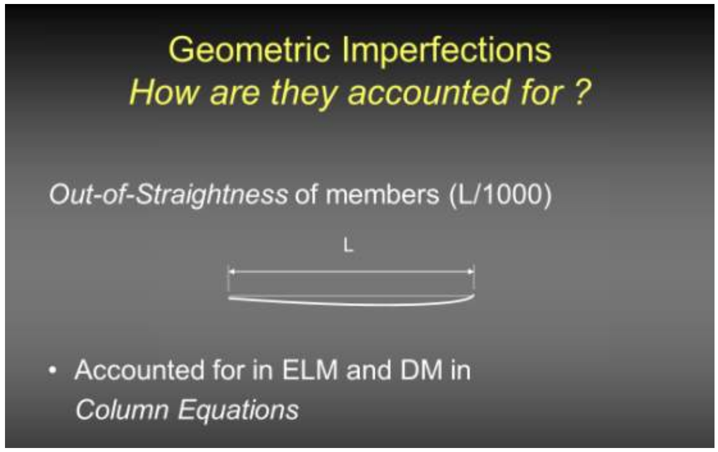
- System
  - Reduced stiffness
  - Additional load
  - Amplified load
  - Modeling




NASCC: THE STEEL CONFERENCE 22

22

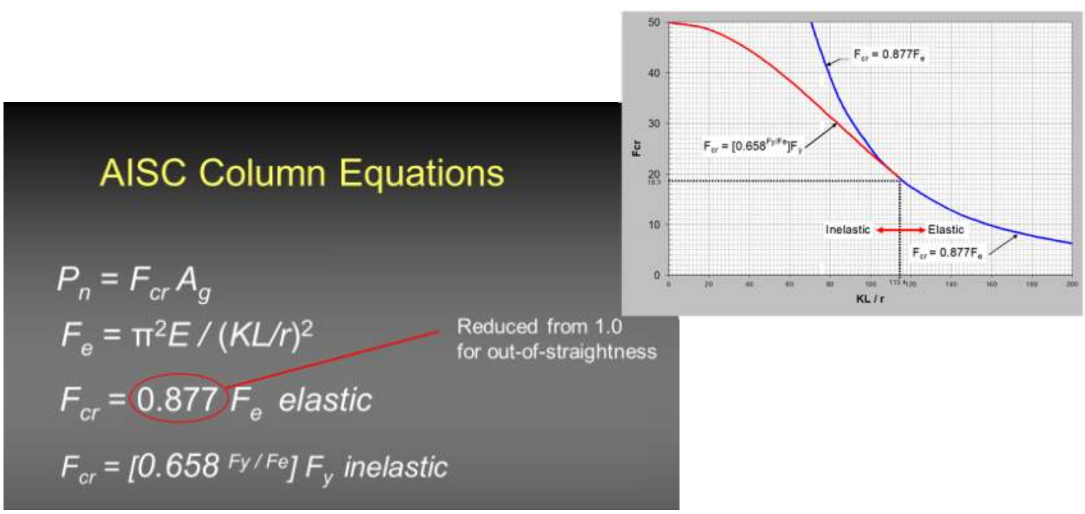
## Accounting for *out-of-straightness*




NASCC: THE STEEL CONFERENCE 23

23

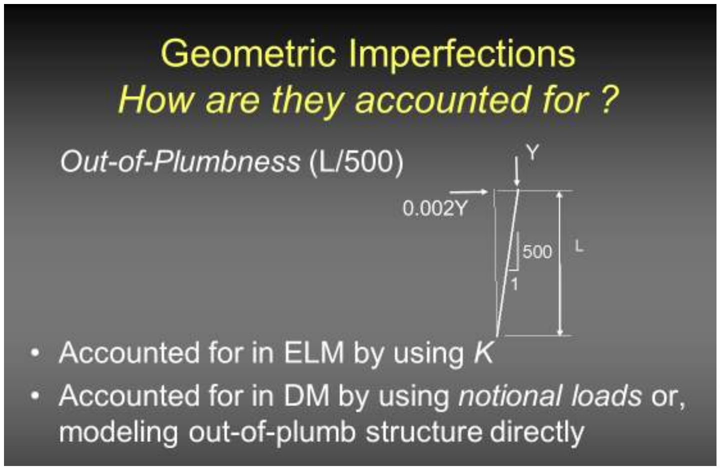
## Accounting for *out-of-straightness*




NASCC: THE STEEL CONFERENCE 24

24

## Accounting for *out-of-plumbness*



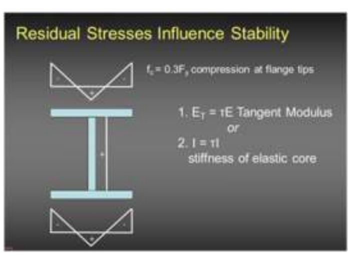

NASCC: THE STEEL CONFERENCE 25

25

## Stiffness reduction due to inelasticity

Residual stress

$M_y$<$M_p$



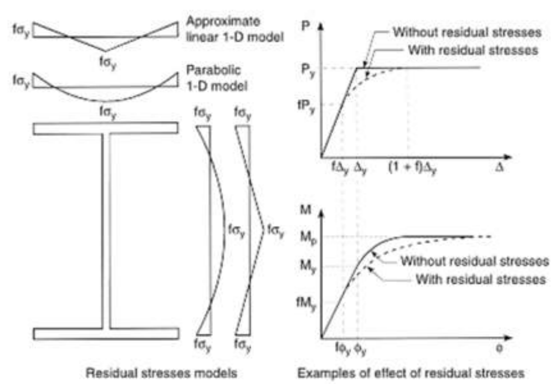




FIGURE 3.4 Schematic representation of self-equilibrating residual stresses in wide-flange structural shape.

NASCC: THE STEEL CONFERENCE 26

26

## First-order (analysis) method: FOM

First-order analysis

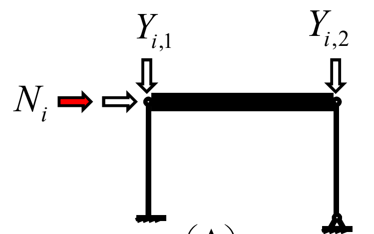
Limit on second-order effects

- $B_2 \leq 1.5$

Additional lateral load

- Based on
  - $B_2 = 1.5$
  - Geometric imperfection
  - Reduction in stiffness due to inelasticity
  - Uncertainty in strength and stiffness

**Use for simple structures**
**Use for quick design check**
**Conservative solutions**




$$N_i = 2.1\alpha\left(\frac{\Delta}{L}\right)Y_i \geq 0.042Y_i$$

$$\frac{\Delta}{L} = \frac{1}{500}$$

NASCC: THE STEEL CONFERENCE

27

27

## The ELM

- Analyze ideal geometrically perfect elastic structure
- Account for residual stresses & geometric imperfections *implicitly with K (buckling analysis)*
- Calculate *K* > 1 for MF's (or obtain elastic column buckling load $P_e$ from a sidesway buckling analysis) - K = 1.0 for braced frames
- Use the AISC column curve to determine $P_n$ (can be expressed in terms of elastic buckling stress $F_e = P_e / A_g$)

NASCC: THE STEEL CONFERENCE

28

28

## Effective length method: ELM

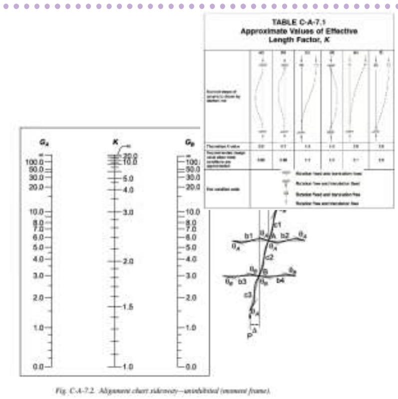
Second-order analysis

Limit on second-order effects

- $B_2 \leq 1.5$

Reduction in moment-frame column strength

- $K > 1.0$ (must determine $K$)
  - Modify for leaning columns
- Addresses stiffness reduction



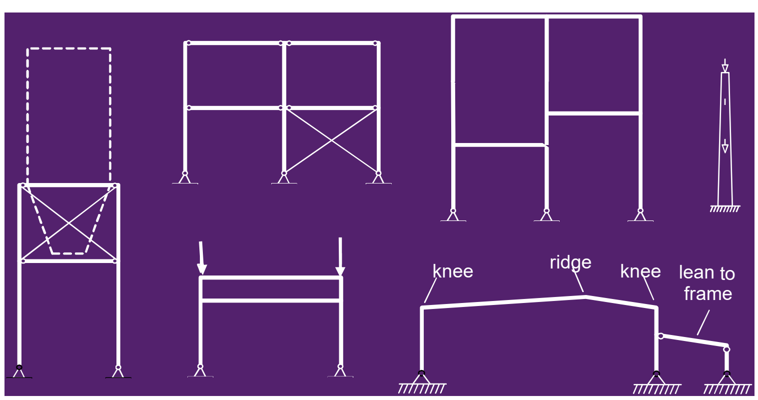

$$K_2 = \sqrt{\frac{\frac{\pi^2 EI}{L^2}}{P_r}\left[\frac{P_{story}}{\Sigma \frac{\pi^2 EI}{(K_{n2}L)^2}}\right]} \geq \sqrt{\frac{5}{8}}K_{n2}$$

NASCC: THE STEEL CONFERENCE

29

29

## Finding *K* can be complicated!

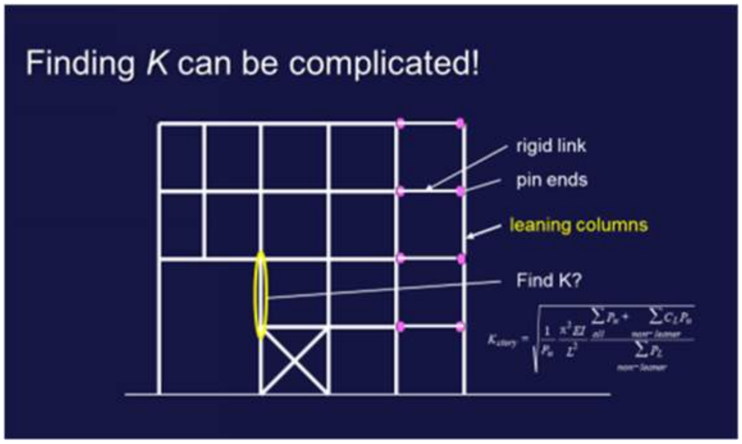



NASCC: THE STEEL CONFERENCE

30

30

## The *K* Factor

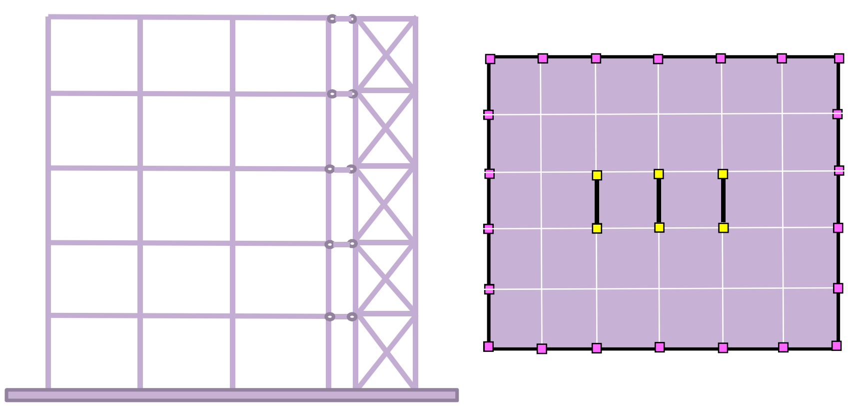



NASCC: THE STEEL CONFERENCE

31

31

## Moment Frame–Braced Frames combined



K = ??

NASCC: THE STEEL CONFERENCE

32

32

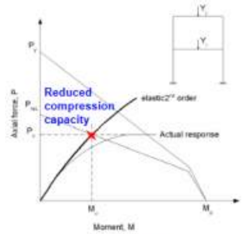
## Direct analysis method: DM

Second-order analysis

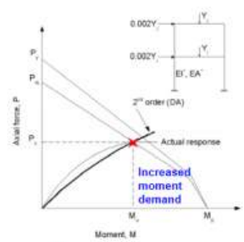
Reduced-stiffness model

- Stiffness reduction due to inelasticity
- Uncertainty in strength and stiffness

$K = 1!$




Effective Length Method




Direct Analysis Method

NASCC: THE STEEL CONFERENCE

33

33

## Direct analysis method: DM

Stiffness reduction

- General
  - $0.8EI$
  - $0.8EA$
- Flexural columns $\alpha P_r/P_{ns} \geq 0.5$ (50% of yield)
  - $0.8\tau_b EI$

$K = 1!$

$$\tau_b = 4\frac{\alpha P_r}{P_{ns}}\left(1 - \frac{\alpha P_r}{P_{ns}}\right) \leq 1.0$$

- Not applicable to
  - *Building period*
  - *Drift*

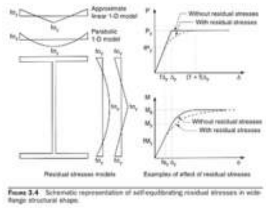
NASCC: THE STEEL CONFERENCE

34

34

## Direct analysis method: DM

Stiffness reduction



**$K$ = 1!**

- General
  - $0.8EI$
  - $0.8EA$
- Flexural columns $\alpha P_r / P_{ns} \geq 0.5$ (50% of yield)
  - $0.8\tau_b EI$

$$\tau_b = 4 \frac{\alpha P_r}{P_{ns}} \left(1 - \frac{\alpha P_r}{P_{ns}}\right) \leq 1.0$$

- Not applicable to
  - *Building period*
  - *Drift*

NASCC: THE STEEL CONFERENCE 35

35

## ELM vs DM
## Interaction Beam Column Equation

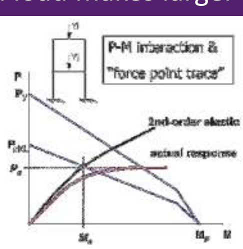
$$for\ \frac{P_r}{P_c} \geq 0.2 \qquad \frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$$

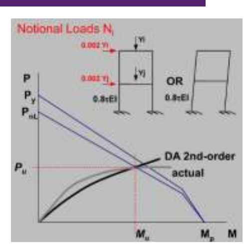
H1-1a

ELM Use of *K>1* causes smaller $P_c$, larger $P_r / P_c$ ***Axial dominates***

DM *K=1* causes larger $P_c$, smaller $P_r / P_c$
notional load makes larger $M / M_c$ ***Moment dominates***

Effective Length Method



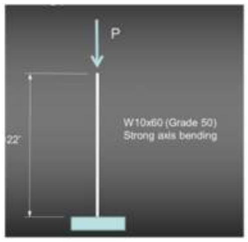



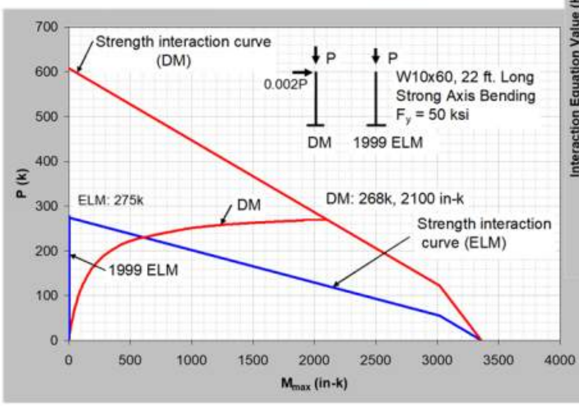

Direct Analysis Method

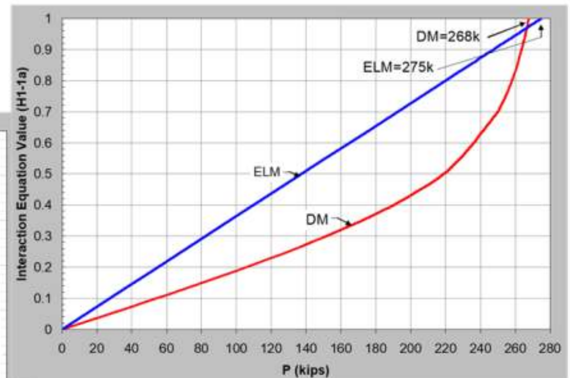
NASCC: THE STEEL CONFERENCE 36

36

## ELM vs DM




Note *non-linear* response in DM

NASCC: THE STEEL CONFERENCE

37

37

## Fundamental differences: ELM vs DM

- Geometric imperfections
  - (out-of-plumbness) included *explicitly* in the DM analysis using *notional loads* or *modeling out-of-plumb geometry*
- *Reduced stiffness* of structure used in DM analysis (accounts for softening of the structure at ultimate load from residual stresses)

NASCC: THE STEEL CONFERENCE

38

38

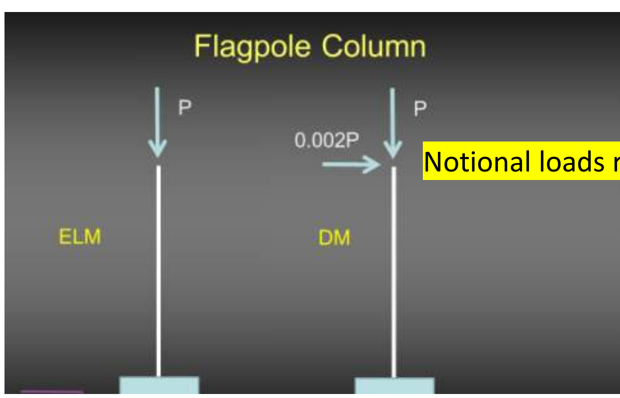
## Major advantages of the Direct Analysis Method

- **No *K* factors** are required!
- Internal forces are more accurate
- Applies to all frame types – moment frames, braced frames, combined systems
- More economical beam-column proportions in certain cases
- The DM now the preferred method in Chapter C

NASCC: THE STEEL CONFERENCE 39

39

## ELM vs DM – Modeling difference



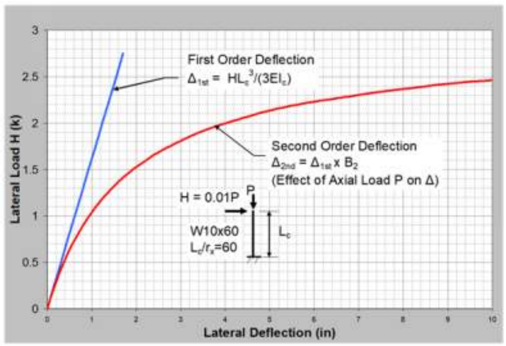

Notional loads required

NASCC: THE STEEL CONFERENCE 40

40

## Second-Order Analysis is important

- Requires input of gravity loads into analysis model
- *Without gravity loads in model, 2nd order analysis is the same as 1st order analysis!*



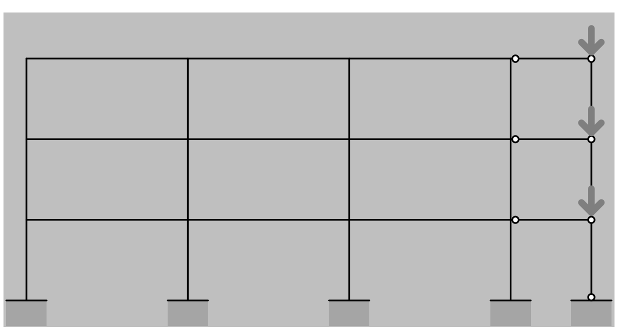

NASCC: THE STEEL CONFERENCE

41

41

## Include **ALL** gravity & other loads that influence the structure's stability

- … the loads in any gravity columns, walls, etc. that are stabilized by the lateral load resisting system must be included in the analysis
- This is often handled by the use of a "dummy column"



NASCC: THE STEEL CONFERENCE

42

42

## Notional loads also apply to the ELM now

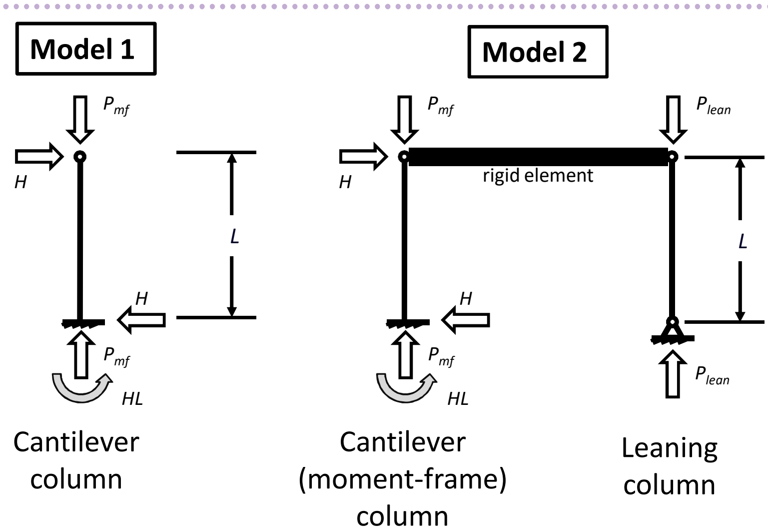
**New restrictions on the ELM:**

- Sidesway amplification limited to 1.5
  - ($\Delta_{2nd}$ / $\Delta_{1st} \leq$ 1.5 based on the nominal unreduced stiffness or
  - $\Delta_{2nd}$ / $\Delta_{1st} \leq$ 1.71 based on the reduced stiffness )
- A notional minimum lateral load is required in gravity-only load combinations (to account for the effect of nominal out-of-plumbness)

NASCC: THE STEEL CONFERENCE 43

43

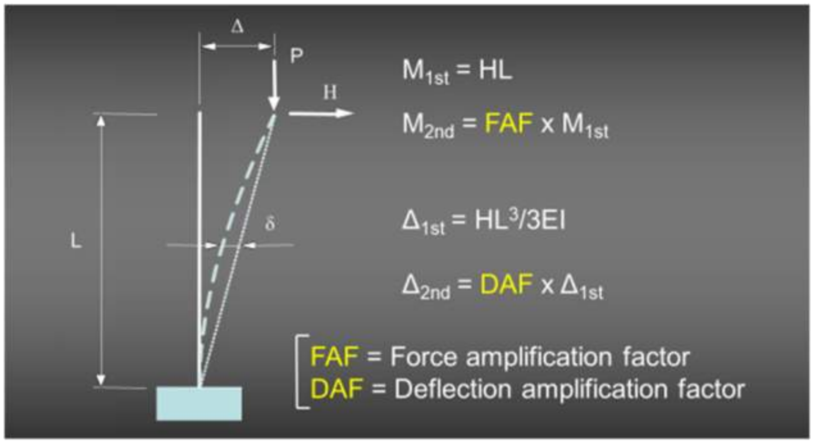
## Simple Stability Models – Useful Tools




NASCC: THE STEEL CONFERENCE 44

44

## Cantilever column – Model 1

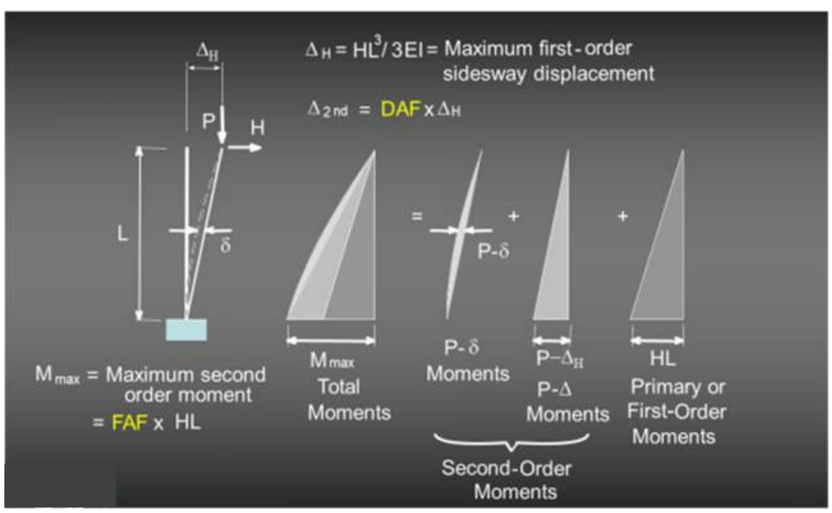



NASCC: THE STEEL CONFERENCE

45

45

## Cantilever column – Model 1

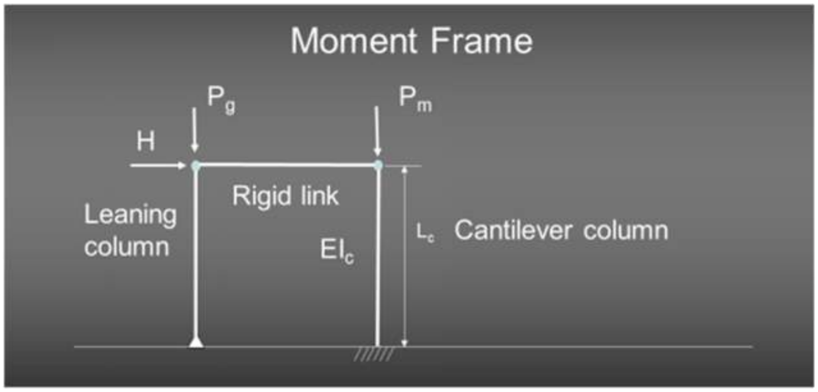



NASCC: THE STEEL CONFERENCE

46

46

## Basic Stability Model – Model 2 (LeMessurier 1977)

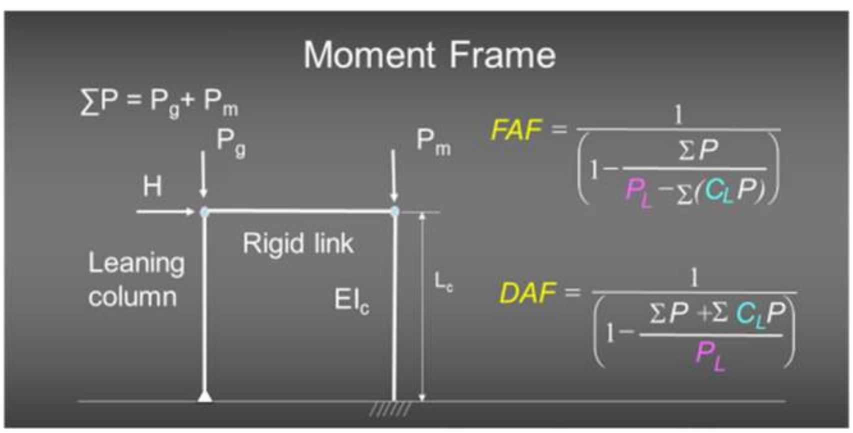



NASCC: THE STEEL CONFERENCE

47

47

## Stability Model 2 (LeMessurier – 1977)



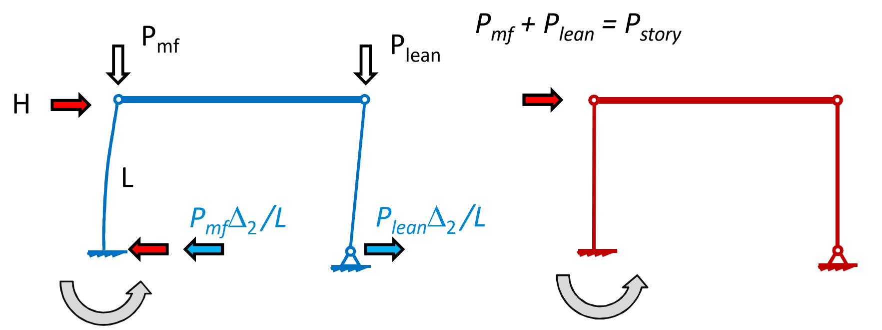

FAF ≈ DAF but not exactly the same!

NASCC: THE STEEL CONFERENCE

48

48

## Second-order analysis



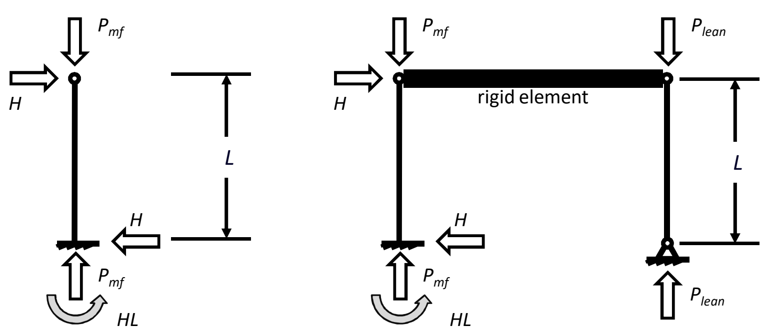

$$HL+\Sigma P\Delta_2 \quad = \quad B_2HL$$

$$B_2 = \frac{HL + P_{story}\Delta_2}{HL} = 1 + \frac{P_{story}\Delta_2}{HL}$$

NASCC: THE STEEL CONFERENCE 49

49

## First-order analysis

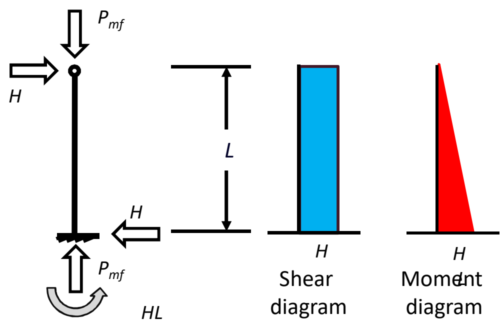



*Inclusion of gravity load does not affect first-order analysis*

NASCC: THE STEEL CONFERENCE 50

50

## First-order analysis

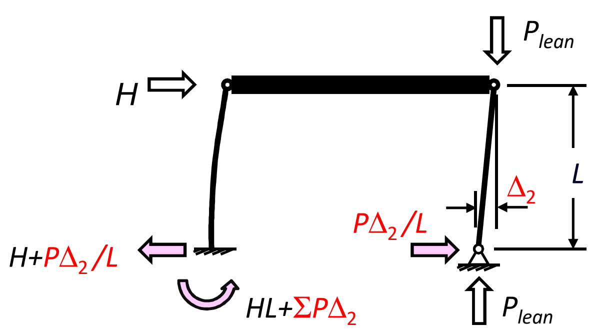



Inclusion of gravity load does not affect first-order analysis

NASCC: THE STEEL CONFERENCE 51

51

## Second-order analysis




Leaning-column effect

Equilibrium in the deformed condition ($\Delta_2$)

NASCC: THE STEEL CONFERENCE 52

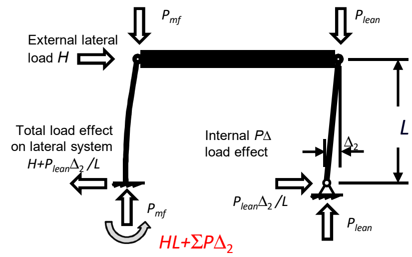
52

## Second-order analysis

Lateral-load-effect amplifier:

$HL+\Sigma P\Delta_2 = B_2 HL$

Gravity load increases lateral-load effects



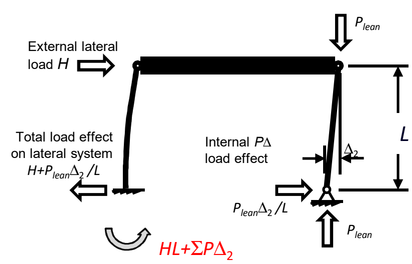

$$B_2 = \frac{HL + \sum P\Delta_2}{HL} = 1 + \frac{\sum P\Delta_2}{HL}$$

NASCC: THE STEEL CONFERENCE

53

53

## Second-order analysis




$$B_2 = 1 + \frac{\sum P\Delta_2}{HL}$$

$\Delta_2 = B_2 \Delta_1$ (true if there is no Pδ effect)

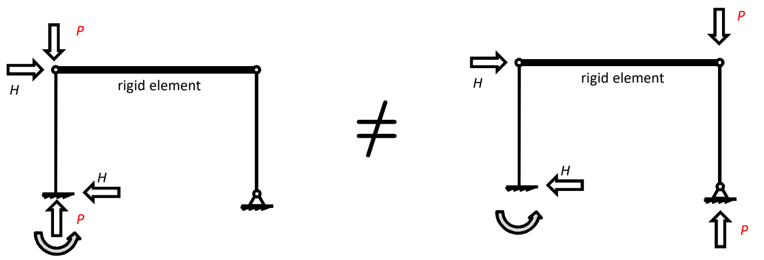
$$B_2 = \frac{1}{1 - \frac{\sum P\Delta_1}{HL}}$$

NASCC: THE STEEL CONFERENCE

54

54

# Pδ stiffness reduction



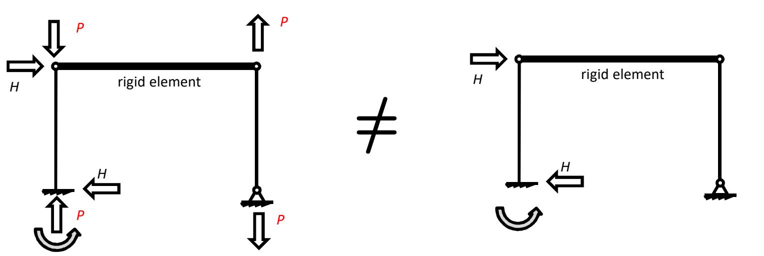

Pδ+PΔ stiffness reduction > PΔ stiffness reduction

Important for moment-frame structures

NASCC: THE STEEL CONFERENCE 55

55

# Pδ stiffness reduction




(Pδ+PΔ)–PΔ stiffness reduction > zero

*Important for moment-frame structures*

NASCC: THE STEEL CONFERENCE 56

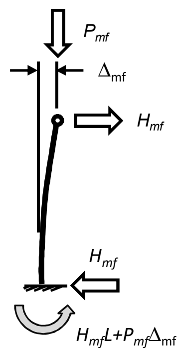
56

## Pδ stiffness reduction

$$B_2 = \frac{1}{1 - \dfrac{P_{story}}{HL/\Delta_1 - \left(12/\pi^2 - 1\right) P_{mf}}} \leq \frac{1}{1 - \dfrac{P_{story}\Delta_1}{\left(1 - 0.15\, P_{mf}/P_{story}\right) HL}}$$

LeMessurier, 1977 AISC *Specification*

Note: AISC parameter $R_M$ is a conservative value appropriate for force amplification




Gravity load on flexural columns has greater effect than gravity load on leaning columns

- Moment-frame columns
- Cantilever columns

NASCC: THE STEEL CONFERENCE

57

57

## Importance of the "leaning columns"

*47 Story Office Building*
*Houston, Texas*

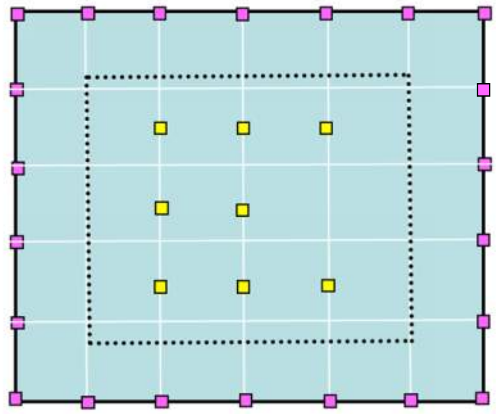
NASCC: THE STEEL CONFERENCE

58

58

## Effect of "leaning columns"

Multistory Office Building



$\sum P$ = all gravity load

$P_{ext} = 0.45\sum P$

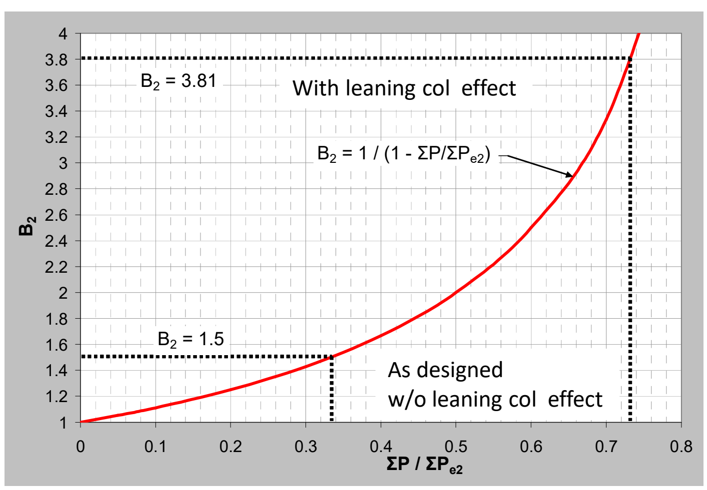
$P_{int} = 0.55\sum P$

MF - Lateral Load Resisting System around building perimeter (30' bays)



59

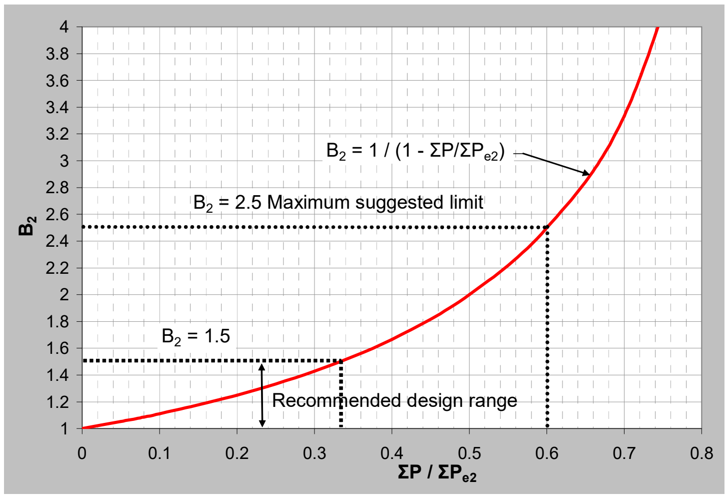
## Effect of "leaning columns"






60

## Recommended range of $B_2$ for design




NASCC: THE STEEL CONFERENCE

61

61

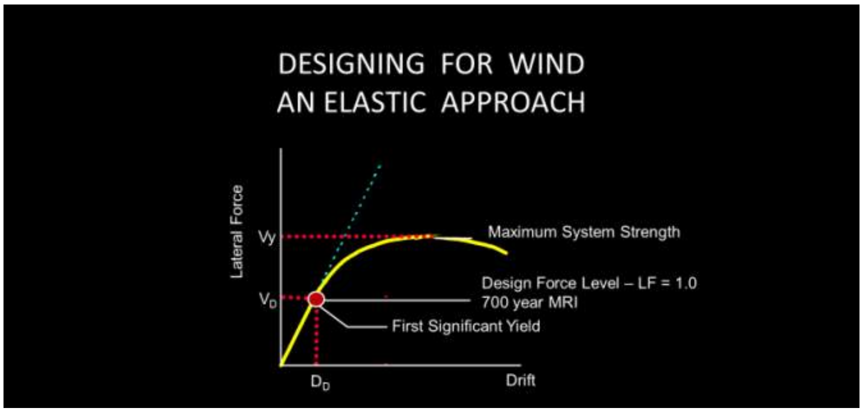
# Limit States - including Drift Control

Limit State 1: Strength under *ultimate wind*
Limit State 2: Serviceability under *service level wind*
Limit State 3: Strength under *design seismic*
Limit State 4: Stability (Drift) under *design seismic*
Limit State 5: Serviceability under *service level seismic*

NASCC: THE STEEL CONFERENCE

62

## Designing for wind load

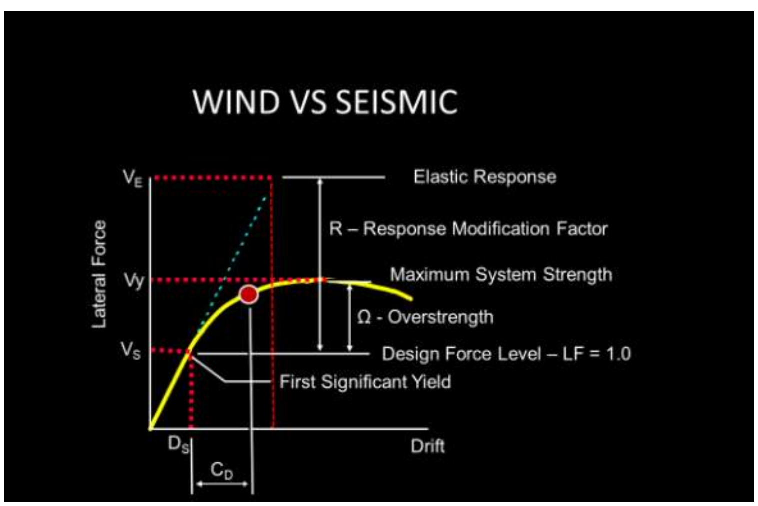



NASCC: THE STEEL CONFERENCE

63

63

## Wind vs Seismic




NASCC: THE STEEL CONFERENCE

64

64

## Design Limit States

LIMIT STATES
TALL BUILDING DESIGN

- Strength Limit State
  - Prescribed by Building Codes – life-safety
  - Strong enough to resist gravity, wind, seismic loads
- Serviceability Limit State
  - Story Drift (Deflection under lateral load)
  - Perception to motion under lateral load “sway” (measured by acceleration in milli-g’s)
  - Engineering judgment (not a code requirement)

NASCC: THE STEEL CONFERENCE 65

65

## Some key factors for building design

THREE KEY FACTORS

- *Strength* under wind and seismic loads
- *Stiffness* to control interstory drift (deflection)
- *Dynamic Properties* (mass, stiffness, damping) to control perception to wind motion

NASCC: THE STEEL CONFERENCE 66

66

## Designing for Wind

CURRENT APPROACH
DESIGNING FOR WIND

Limit States (Performance Levels):

- Strength: MRI = 700, 1700 Years (Code)
- Story Drift: MRI = 10, 25, 50,100 Years (Engr. Judgment)
- Perception to Motion: MRI = 1,10 Years (Engr. Judgment)

NASCC: THE STEEL CONFERENCE 67

67

## The importance of the wind tunnel

If you want to understand how
real buildings behave
under wind storms……….

*Go to the Wind Tunnel!*

frequencies,
mode shapes,
mass,
stiffness,
damping
"True Building Response"

NASCC: THE STEEL CONFERENCE 68

68

## Damage control under wind load


NASCC: THE STEEL CONFERENCE

69

69

## Serviceability Limit States can control design

SERVICEABILITY LIMIT STATES

- ***IMPORTANT NOTE:***
- *SERVICEABILITY LIMIT STATES (e.g. deflections, vibration) often control the design of many building floor members – particularly in lighter steel framed buildings.*
- *Many designers will design for serviceability limit states first – and then check strength limit states as a final step in the design.*

NASCC: THE STEEL CONFERENCE

70

70

## Requirements for assessing building drift & damage control

- Build an accurate analysis model
- Define one or more serviceability load combinations
- Define a damage measure that captures the damage potential from building sway

NASCC: THE STEEL CONFERENCE

71

71

## The change in member properties *Serviceability* limit state to *Strength* limit state load levels

- GROSS MEMBER PROPERTIES – under serviceability load combinations, structure remains essentially elastic
- REDUCED MEMBER PROPERTIES, reduced stiffness, under strength limit states, structures in inelastic range of response
- Applies to steel structures, more so to concrete structures
- Applies to building Period T
- Applicable in wind tunnel studies
- **Must consider this effect in design**

**TWO SEPARATE MODELS – ONE FOR EACH LIMIT STATE**

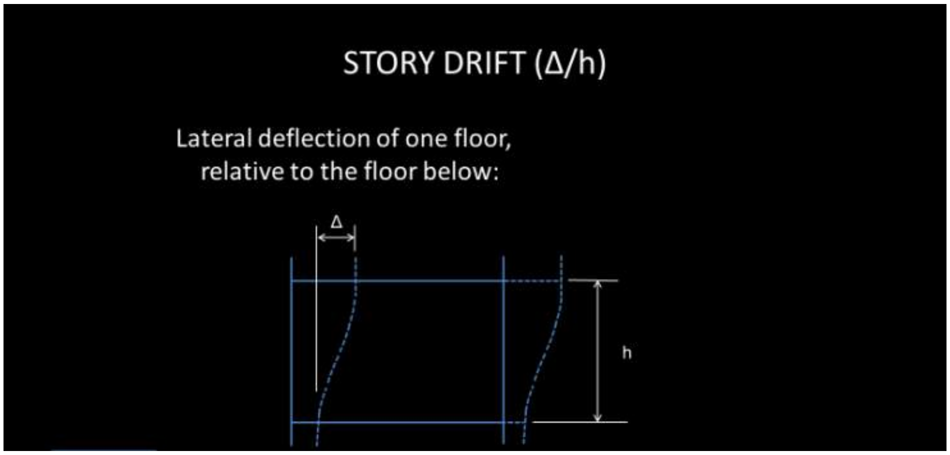
*A $B_3$ Factor can change this!*

NASCC: THE STEEL CONFERENCE

72

72

## Story Drift – a popular measure of deflection control

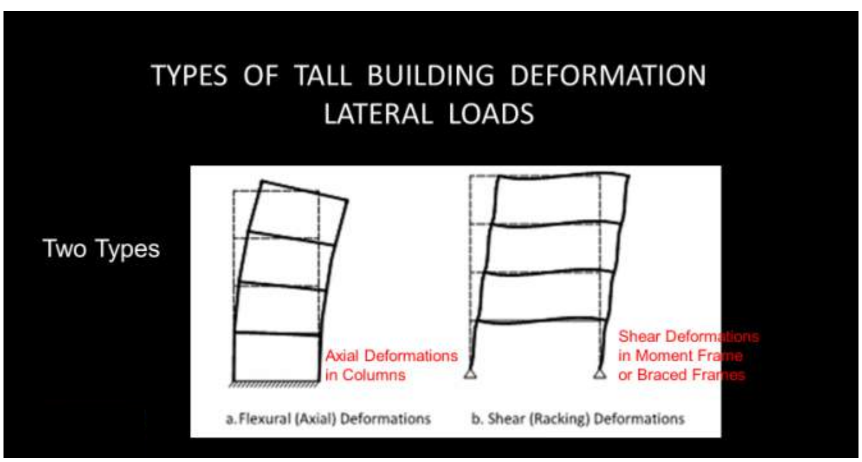



NASCC: THE STEEL CONFERENCE

73

73

## Deformation components

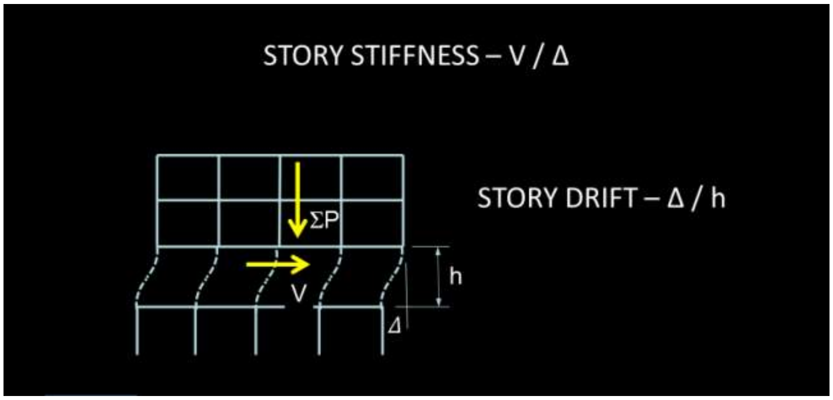



NASCC: THE STEEL CONFERENCE

74

74

## "Racking" deformation



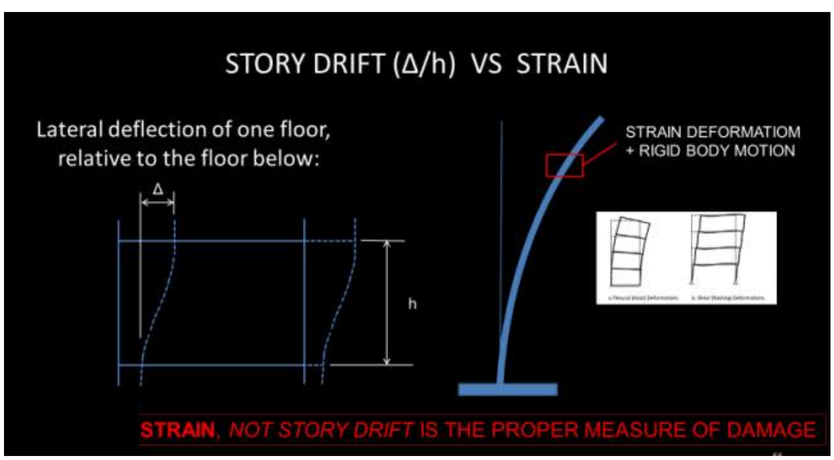

*Important for moment frame systems with wide column spacing*

NASCC: THE STEEL CONFERENCE

75

75

## Story Drift vs Strain

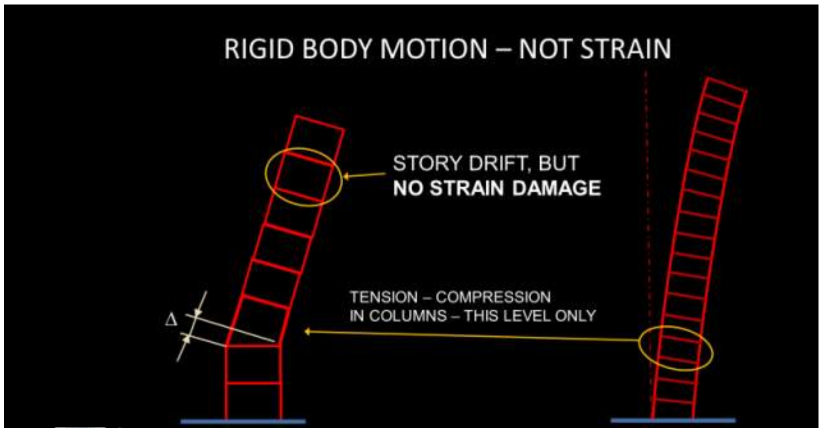



NASCC: THE STEEL CONFERENCE

76

76

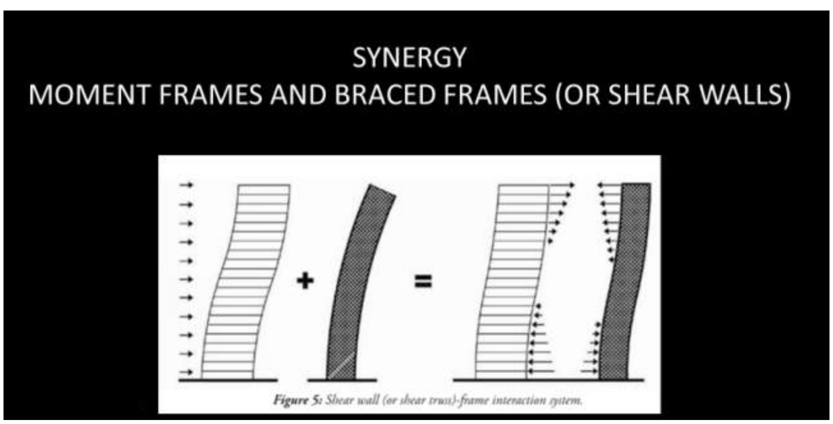
## Rigid body motion *does not cause strain*




NASCC: THE STEEL CONFERENCE 77

77

## Moment Frame – Braced Frame interaction




NASCC: THE STEEL CONFERENCE 78

78

## Designing for *Drift* and *Perception to Motion*

THE NEW YORK SLENDER TOWER STORY
CONDOS -*BILLIONAIR'S ROW*

111 W 57th ST
84 stories,
1428 ft
h/w = 23/1

435 Park Av.
85 stories,
1396 ft
h/w = 15/1

NASCC: THE STEEL CONFERENCE

79

79

## Strength to Serviceability Ratio

STRENGTH VS SERVICEABILITY FORCES/DEFLECTIONS EXAMPLES

- Kansas City – Typical inland city (middle of range US wind city)
  $V_{700}$ = 110 MPH; $V_{25}$ = 79 MPH; $V^2$ Ratio = 1.94
- Los Angeles – West coast city (lower bound US wind city)
  $V_{700}$ = 115 MPH; $V_{25}$ = 84 MPH; $V^2$ Ratio = 1.87
- Miami – High Hurricane city (upper bound range US wind city)
- $V_{700}$ = 170 MPH; $V_{25}$ = 113 MPH; $V^2$ Ratio = 2.26

| STR / SERV (700/25) | LOS ANGELES | KANSAS CITY | MIAMI |
|---|---|---|---|
| FORCE - CONCRETE | 2.24 | 2.33 | 2.71 |
| LATERAL Δ - CONCRETE | 3.19 | 3.31 | 3.85 |
| FORCE - STEEL | 2.24 | 2.33 | 2.71 |
| LATERAL Δ - STEEL | 2.80 | 2.91 | 3.39 |

NASCC: THE STEEL CONFERENCE

80

80

## Story Drift

STORY DRIFT (Δ/h)

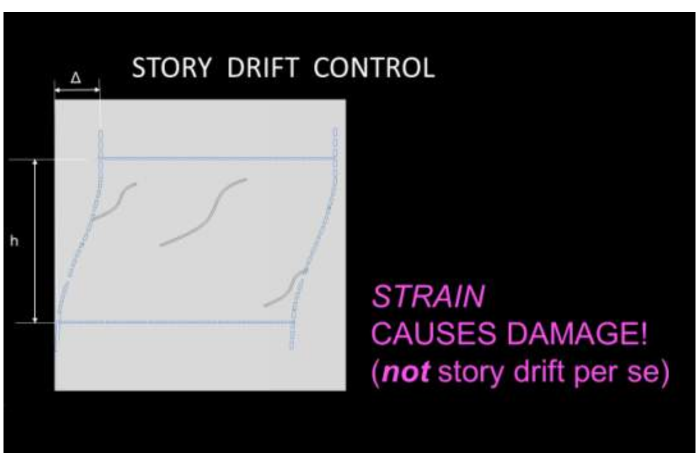
- Choose demand wind storm
  - 10, 25, 50, 100 yr MRI (ASCE 7 App C)
- Choose Drift Limit (material, cladding type)
  - h/100 – h/800 limits common (1/400 = 0.0025)
- Accurate computer model
  - Ultimate, service level
  - Proper stiffness selection (cracking)
  - Consider all deformations applicable (axial, flexural, shear, panel zone, P-Δ effects)

NASCC: THE STEEL CONFERENCE

81

81

## Story Drift vs Strain

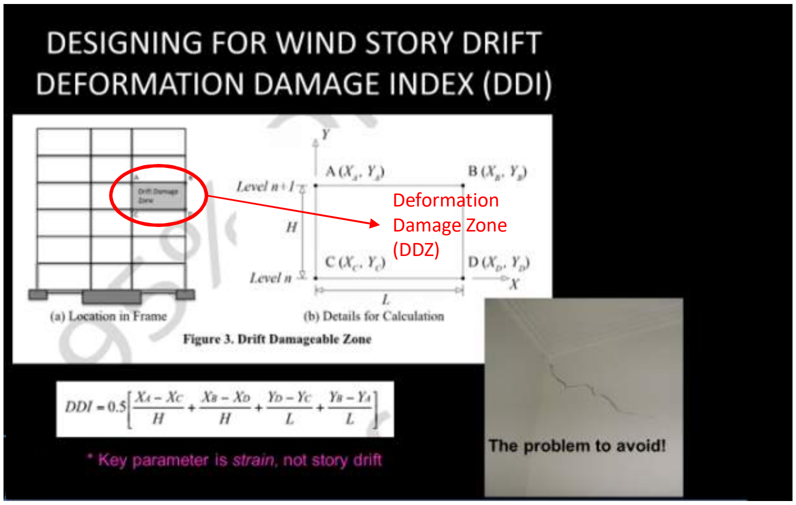



NASCC: THE STEEL CONFERENCE

82

82

## It's really a matter of *strain* – ***not** drift*

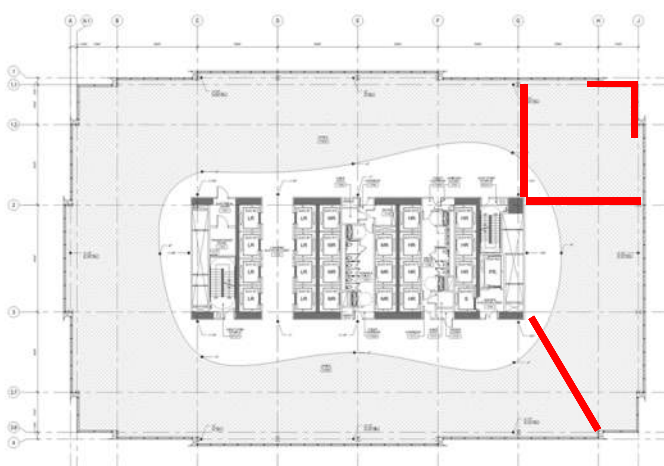



NASCC: THE STEEL CONFERENCE

83

83

## Deformation Damage Zones (DDZ)



Suggested locations,
Warping of floor slabs occurs

NASCC: THE STEEL CONFERENCE

84

84

## Suggested Drift Design Criteria – ATC Design Guide

### SUGGESTED DRIFT DESIGN CRITERIA DEFORMATION DAMAGE INDEX (DDI)

Table 2

Suggested Minimum Building Serviceability (Drift) Criteria under Wind Load

| Building Quality and Durability | Serviceability Wind Load | DDI (Strain) | Comment |
|---|---|---|---|
| Minimum | 10 yr. MRI | 0.0025 | Building defined by Stakeholders desire for a *minimum* standard level of quality and durability at least cost. |
| Normal | 25 yr. MRI | 0.0025 | Building defined by Stakeholders desire for a *typical or mid-level* standard of quality and durability. |
| High | 50 yr. MRI | 0.002 | Building defined by Stakeholders desire for a *high level* standard of quality and durability. |
| Premium | 100 yr. MRI | 0.002 | Building defined by Stakeholders desire for a *premium level* standard of quality and durability |

Irwin, Peter
Griffis, Larry





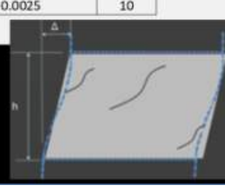
## The Deformation Damage Index Limits


### DEFORMATION DAMAGE INDICES (DDI)

Table 3. Suggested DDI Limits for Serviceability Design

| | Building Element | Suggested DDI Limit | Notes |
|---|---|---|---|
| Exterior Cladding | Brick Veneer w/ metal studs | 0.0025 | 1 |
| | Brick Veneer w/ unreinf. masonry | 0.0025 | 1,2 |
| | Plaster/Stucco | 0.0025 | 3 |
| | Architectural Precast | 0.0025 | 4 |
| | Stone Clad Precast | 0.0025 | 4 |
| | Architectural Metal Panel | 0.01 | 5 |
| | Curtain Wall, Window Wall | 0.0025 | 6 |
| Interior Partitions | Gypsum Drywall, Plaster | 0.0025 | 7 |
| | Concrete Masonry, unreinf. | 0.00150 | 8 |
| | Tile, Hollow Clay Brick | 0.0005 | 9 |
| Elevators | Drywall enclosure | 0.0025 | 10 |

The problem to avoid!




### FRAGILITY CURVE (FEMA P-58)

Fragility Curve – a mathematical relationship between an engineering demand parameter (e.g. shear strain in a nonstructural component) and the probability of attaining damage

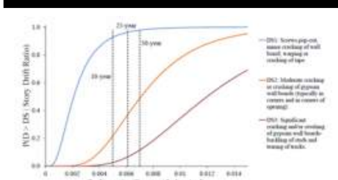
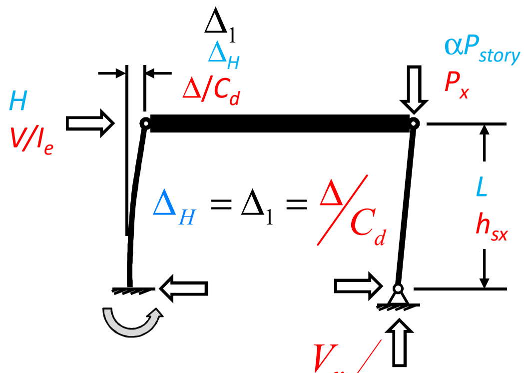


Figure 7. Gypsum Wall Board Fragility Curves (PACT)

## AISC 360/341 vs. ASCE-7 symbols




$$B_2 = \frac{1}{1 - \theta / R_M}$$

$$B_2 = \frac{1}{1 - \dfrac{\alpha P_{story} \Delta_H}{R_M H L}}$$

$$K_M = \frac{H}{\Delta_H} = \frac{V_x / I_e}{\Delta / C_d}$$

Mechanical stiffness

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d}$$

NASCC: THE STEEL CONFERENCE

87

87

## Seismic design for drift and stability – ASCE 7-16

- ASCE 7-16 Section 12.12 Drift and Deformation
- Table 12.12-1 Allowable Story Drift $\Delta_{all}$

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d} \quad (12.8\text{-}16)$$

**STABILITY INDEX θ**

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25 \quad (12.8\text{-}17)$$

NASCC: THE STEEL CONFERENCE

88

88

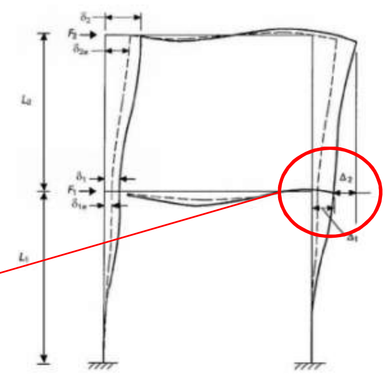
## Seismic design for drift and stability – ASCE 7-16

- Design Story Drift Δ:

$$\delta_x = \frac{C_d \delta_{xe}}{I_e} \qquad (12.8\text{-}15)$$

$\delta_2 = C_d \delta_{2e} / I_E$ = amplified displacement; .
(Table 12.12-1).

$$\Delta_2 = C_d(\delta_{2e} - \delta_{1e})/I_E \le \Delta_a$$



***Drift Control*** under ultimate design loads

NASCC: THE STEEL CONFERENCE

89

89

## Seismic Drift Limits – ASCE 7 Table 12.12-1

**Table 12.12-1 Allowable Story Drift, $\Delta_a$[a,b]**

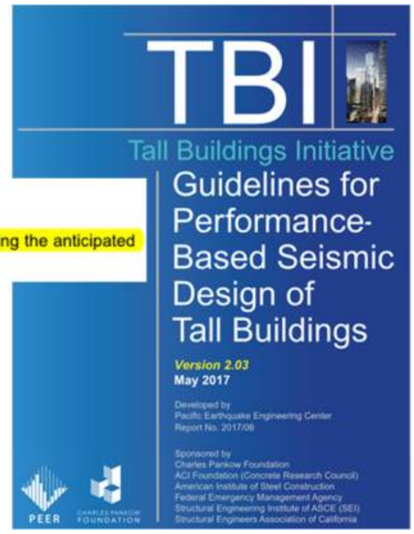
| Structure | Risk Category: I or II | III | IV |
|---|---|---|---|
| Structures, other than masonry shear wall structures, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts | $0.025h_{sx}$[c] | $0.020h_{sx}$ | $0.015h_{sx}$ |
| Masonry cantilever shear wall structures[d] | $0.010h_{sx}$ | $0.010h_{sx}$ | $0.010h_{sx}$ |
| Other masonry shear wall structures | $0.007h_{sx}$ | $0.007h_{sx}$ | $0.007h_{sx}$ |
| All other structures | $0.020h_{sx}$ | $0.015h_{sx}$ | $0.010h_{sx}$ |

NASCC: THE STEEL CONFERENCE

90

90

## Performance Based Design
## The Analysis Model




**4.6 STRUCTURAL MODELING PARAMETERS**

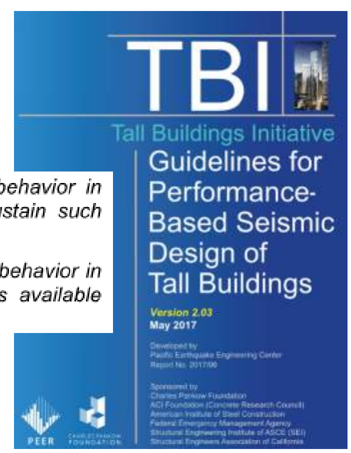
Develop the structural analysis model based on expected properties considering the anticipated level of response and damage.

NASCC: THE STEEL CONFERENCE

91

91

## Performance Based Design – Seismic Tall Buildings




***Deformation-controlled action*** *– An action expected to undergo nonlinear behavior in response to earthquake shaking, and which is evaluated for its ability to sustain such behavior.*

***Force-controlled action*** *– An action that is not expected to undergo nonlinear behavior in response to earthquake shaking, and which is evaluated on the basis of its available strength.*

NASCC: THE STEEL CONFERENCE

92

92

## Structural Analysis Model

**4.6 STRUCTURAL MODELING PARAMETERS**

Develop the structural analysis model based on expected properties considering the anticipated level of response and damage.

> ***Commentary**: The structural analysis model is intended to provide an unbiased, best estimate of expected response of the building when subjected to earthquake ground motion. For this reason, the structural analysis model should be developed based on expected material and member stiffnesses, strengths, and deformation capacities, rather than values that have been adjusted to achieve a "conservative" estimate of response.*

NASCC: THE STEEL CONFERENCE 93

93

## Service Level Earthquake – PEER (MRI = 43 years)

Service-Level Earthquake (SLE) shaking.

**LINEAR DYMAMIC ANALYSIS**

- Response Spectrum Analysis
- Response History Analysis

Define SLE shaking as required in Chapter 3, with a minimum return period of 43 years (50% probability of exceedance in 30 years). Represent SLE shaking in the form of a site-specific, uniform hazard acceleration response spectrum, with damping as indicated in Section 4.2.7. If nonlinear response history analysis is to be performed as part of the SLE evaluation, select ground motions and modify them to be compatible with the SLE spectrum in accordance with the recommendations of Chapter 3.

> ***Commentary:** SLE shaking is typically set at a return period of 43 years. Consequently, it can be reasonably expected that a tall building will be subjected to earthquake shaking at or exceeding this shaking level once or more during its service lifetime. This expectation has been taken into consideration in establishing the SLE criteria.*
>
> *Considering that intended response of the building to SLE shaking is essentially elastic, the*

NASCC: THE STEEL CONFERENCE 94

94

## Service Level EQ – Load Combinations

**5.5.1 Load Combinations: Linear Modal Response Spectrum Analysis**

Evaluate the structure for strength and drift for the following load combinations:

$$1.0D + 0.5L + 1.0E_X + 0.3E_Y \quad (5\text{-}1)$$

$$1.0D + 0.5L + 0.3E_X + 1.0E_Y \quad (5\text{-}2)$$

Take live load, *L*, *as* 80% of unreduced live loads that exceed 100 pounds per square foot (4.79 kN/m$^2$) and 40% of other unreduced live loads.

NASCC: THE STEEL CONFERENCE

95

95

## Seismic Serviceability – Story Drift Limits

**5.6.1 Story Drift Limit**

Calculated story drift shall not exceed 0.5% of story height in any story, computed at extreme points for each floor plan in each of two orthogonal plan directions.

> ***Commentary**: The story drift limit of 0.5% for SLE shaking is intended to provide some protection of nonstructural components and also to ensure that permanent lateral displacement of the structure will be negligible. Extreme points on the floor plans are evaluated to help ensure that torsional response is properly controlled. It is important to understand that at a story drift of 0.5%, nonstructural damage, particularly for elements such as interior partitions, may not be negligible and considerable cosmetic repair may be required. Evaluation of damage to nonstructural elements can be performed using tools such as FEMA P58.*

NASCC: THE STEEL CONFERENCE

96

96

## Deformation Controlled Actions

**5.7 COMPONENT ACCEPTANCE CRITERIA–LINEAR ANALYSIS**

**5.7.1 Deformation-Controlled Actions**

When response spectrum or linear response history analysis is used for the SLE evaluation, calculated demand-to-capacity ratios for deformation-controlled actions shall not exceed 1.5, where demand is calculated from load combinations in accordance with Section 5.5, and capacity is calculated as follows:

1. For reinforced concrete elements, the capacity is defined as the nominal strength in accordance with ACI 318 without applying the corresponding strength reduction factor $\phi$.
2. For structural steel and composite steel and concrete elements, the capacity is defined as the nominal LRFD strength in accordance with AISC 341 and AISC 360, which is taken as the nominal strength without applying the corresponding resistance factor $\phi$.

*anticipated that expected strengths will be higher than the nominal strengths. Consequently, the demand-to-capacity ratio of 1.5 based on design strengths can be expected to result in only minor inelastic response.*

*These Guidelines do not provide more restrictive requirements for Risk Category III buildings in the SLE evaluation since the building code focuses primarily on limiting the probability of collapse for that Risk Category. If a higher level of certainty of meeting the service-level performance goals is desired for Risk Category III and IV buildings, a lower demand-to-capacity ratio for the deformation-controlled actions (for example, 1.5/1.25 = 1.2 for Risk Category III buildings) can be applied.*

NASCC: THE STEEL CONFERENCE 97

97

## Force Controlled Actions

**5.7.2 Force-Controlled Actions**

Calculated demand-to-capacity ratios for force-controlled actions shall not exceed 1.0, where demand is calculated from load combinations in accordance with Section 5.5, and capacity is calculated as follows:

1. For reinforced concrete elements and their connections, the capacity is defined as the design strength, taken as the nominal strength multiplied by the corresponding strength reduction factor $\phi$ in accordance with ACI 318.
2. For structural steel elements, for composite steel and concrete elements, and for their connections, the capacity is defined as the LRFD strength, which is taken as the nominal strength multiplied by the corresponding resistance factor $\phi$ in accordance with AISC 341 and AISC 360.

***Commentary:*** *For force-controlled actions, strength reduction (resistance) factors $\phi$ of ACI 318, AISC 341, and AISC 360 and a demand-to-capacity ratio of 1.0 have been defined to promote a strength hierarchy in which yielding occurs in deformation-controlled actions before force-controlled actions. However, these factors alone may be insufficient to ensure that an appropriate yielding mechanism occurs. It is not uncommon, therefore, to design a*

NASCC: THE STEEL CONFERENCE 98

98

## Modeling Member Properties – Structural Steel

### 4.6.3 Effective Member Stiffness

Steel members and components: Model the elastic (initial) stiffness of steel members and components using full cross-sectional properties and the elastic modulus of steel [$E_s$ = 29,000 ksi (200,000 MPa)].

Reinforced concrete components: In lieu of detailed justification, use the values in Table 4-3 for the effective stiffness of reinforced concrete members and components, along with component-specific guidance in subsections of Section 4.6.5.

NASCC: THE STEEL CONFERENCE

99

99

## Modeling Member Properties – Reinforced Concrete

**Table 4-3 Reinforced concrete effective stiffness values.**

| Component | Service-Level Linear Models | | | MCE$_R$-Level Nonlinear Models | | |
|---|---|---|---|---|---|---|
| | Axial | Flexural | Shear | Axial | Flexural | Shear |
| Structural walls[1] (in-plane) | $1.0E_cA_g$ | $0.75E_cI_g$ | $0.4E_cA_g$ | $1.0E_cA_g$ | $0.35E_cI_g$ | $0.2E_cA_g$ |
| Structural walls (out-of-plane) | -- | $0.25E_cI_g$ | -- | -- | $0.25E_cI_g$ | -- |
| Basement walls (in-plane) | $1.0E_cA_g$ | $1.0E_cI_g$ | $0.4E_cA_g$ | $1.0E_cA_g$ | $0.8E_cI_g$ | $0.2E_cA_g$ |
| Basement walls (out-of-plane) | -- | $0.25E_cI_g$ | -- | -- | $0.25E_cI_g$ | -- |
| Coupling beams with conventional or diagonal reinforcement | $1.0E_cA_g$ | $0.07\left(\frac{\ell}{h}\right)E_cI_g \le 0.3E_cI_g$ | $0.4E_cA_g$ | $1.0E_cA_g$ | $0.07\left(\frac{\ell}{h}\right)E_cI_g \le 0.3E_cI_g$ | $0.4E_cA_g$ |
| Composite steel / reinforced concrete coupling beams | $1.0(EA)_{trans}$ | $0.07\left(\frac{\ell}{h}\right)(EI)_{trans}$ | $1.0E_sA_{sw}$ | $1.0(EA)_{trans}$ | $0.07\left(\frac{\ell}{h}\right)(EI)_{trans}$ | $1.0E_sA_{sw}$ |
| Non-PT transfer diaphragms (in-plane only)[3] | $0.5E_cA_g$ | $0.5E_cI_g$ | $0.4\ E_cA_g$ | $0.25E_cA_g$ | $0.25E_cI_g$ | $0.1E_cA_g$ |
| PT transfer diaphragms (in-plane only)[3] | $0.8E_cA_g$ | $0.8E_cI_g$ | $0.4E_cA_g$ | $0.5E_cA_g$ | $0.5\ E_cI_g$ | $0.2E_cA_g$ |
| Beams | $1.0E_cA_g$ | $0.5E_cI_g$ | $0.4E_cA_g$ | $1.0E_cA_g$ | $0.3E_cI_g$ | $0.4E_cA_g$ |
| Columns | $1.0E_cA_g$ | $0.7\ E_cI_g$ | $0.4E_cA_g$ | $1.0E_cA_g$ | $0.7\ E_cI_g$ | $0.4E_cA_g$ |
| Mat (in-plane) | $0.8E_cA_g$ | $0.8E_cI_g$ | $0.8E_cA_g$ | $0.5E_cA_g$ | $0.5\ E_cI_g$ | $0.5E_cA_g$ |
| Mat[4] (out-of-plane) | -- | $0.8E_cI_g$ | -- | -- | $0.5E_cI_g$ | -- |

NASCC: THE STEEL CONFERENCE

100

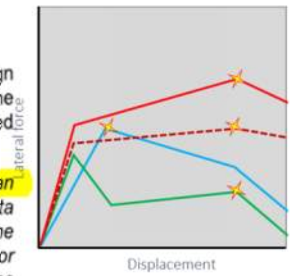
100

## P-Delta Analysis – A *Caution* for Seismic Design

**4.2.8 P-Delta Effects**

Include P-Delta effects in nonlinear analysis, regardless of whether elastic analysis design checks indicate that such effects are important. The P-Delta effects should include the destabilizing gravity loads for the entire building, where the gravity loads are spatially distributed to capture both building translation and twist.

> ***Commentary:*** *The widely used elastic stability coefficient ($\theta = P\delta/Vh$) is often an insufficient indicator of the importance of P-Delta effects in the inelastic range. P-Delta effects may become an overriding consideration when strength deterioration sets in and the tangent stiffness of the story shear force versus story drift relationship approaches zero or becomes negative. When this happens, the story drift ratchets, that is, it increases in one direction without the benefit of a full reversal that otherwise would straighten out the story. For this reason, and many others, realistic modeling of component deterioration and post-yield stiffness are critical aspects of modeling. The potential for dynamic instability is relatively high in flexible moment frame structures and in braced frames and shear wall structures in which one or several of the lower stories deform in a shear mode and the tributary gravity loads are large such that P-Delta will lead to a significant amplification of story drift demands. ATC 72 (2010) Chapter 2 provides detailed information on P-Delta effects and why and when the effects become an important consideration in inelastic structural response.*




Positive slope can prevent instability

Negative slope can lead to instability

NASCC: THE STEEL CONFERENCE

101

101

## Summary

Design for stability is necessary

- Must address "Big Five" considerations, including:
  - $2^{nd}$-order effects
    - – $B_2$ amplifier
      - » Or $B_2$–based force (FOM)
    - – Explicit second-order analysis
  - Stiffness-reduction effects
    - – ELM: *K*
    - – DM: Reduced stiffness ($0.8\tau_b$)
    - – FOM: 0.8–based force; axial-force limit

NASCC: THE STEEL CONFERENCE

102

102

## Summary

Second-order effects are a key part of design for stability

Second-order effects should be included in *all drift* evaluations

Second-order effects can be included in the analysis or addressed via amplifiers ($B_1$, $B_2$ and the new $B_3$)

Inclusion of full system gravity load is necessary to capture PΔ effects

- Select appropriate load combination for reduced stiffness

Proper modeling is necessary to capture Pδ effects

- Mesh members (4 segments) or use the $B_1$ amplifier

NASCC: THE STEEL CONFERENCE

103

103

## Design Amplifiers – You know about two of them

- The AISC $B_1$ Amplifier

$$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1 \quad \text{(A-8-3)}$$

- The AISC $B_2$ Amplifier

$$B_2 = \frac{1}{1 - \frac{\alpha P_{story}}{P_{e\,story}}} \geq 1 \quad \text{(A-8-6)}$$

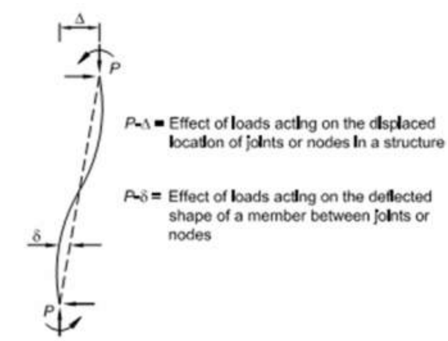



*Fig. C-C2.1. P-Δ and P-δ effects in beam-columns.*

NASCC: THE STEEL CONFERENCE

104

104

## And now, there is a third one – *THE $B_3$ AMPLIFIER*

The ***$B_3$***

***"SABELLI AMPLIFIER"***

$$B_3 = \frac{0.8\tau_b}{1-[1-0.8\tau_b]B_2}$$

NASCC: THE STEEL CONFERENCE

105

105

The ***$B_3$***

***"SABELLI AMPLIFIER"***

$$B_3 = \frac{0.8\tau_b}{1-[1-0.8\tau_b]B_2}$$

**It's TASTY!**

NASCC: THE STEEL CONFERENCE

106

106


NASCC:
THE STEEL CONFERENCE
BRIEF PAUSE…….
RAFAEL COMING UP

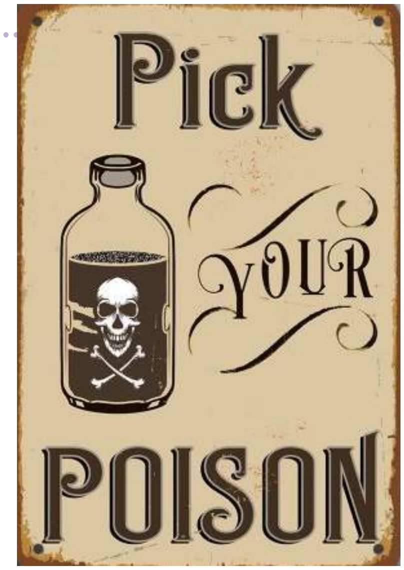
107


NASCC:
THE STEEL CONFERENCE
The Indirect
Analysis Method

108

## Stability-design methods

| | Braced Frames | Moment Frames |
|---|---|---|
| FOM | Additional lateral force<br>Conservative | Additional lateral force<br>Conservative |
| ELM | | *K* factors<br>Adjust for leaning columns |
| DM | Reduced-stiffness model | Reduced-stiffness model |




NASCC: THE STEEL CONFERENCE

109

109

## Indirect analysis method

Based on Direct analysis method

Full-stiffness model

$K$=1

There has to be a catch, right?

- Uses $B_3$ factor instead of reduced stiffness

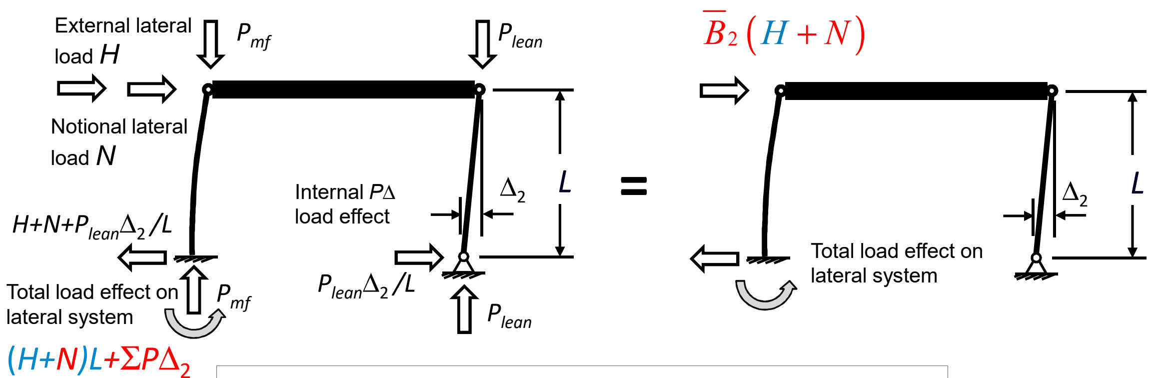


NASCC: THE STEEL CONFERENCE

110

110

## Equilibrium in the deformed condition: amplifier method



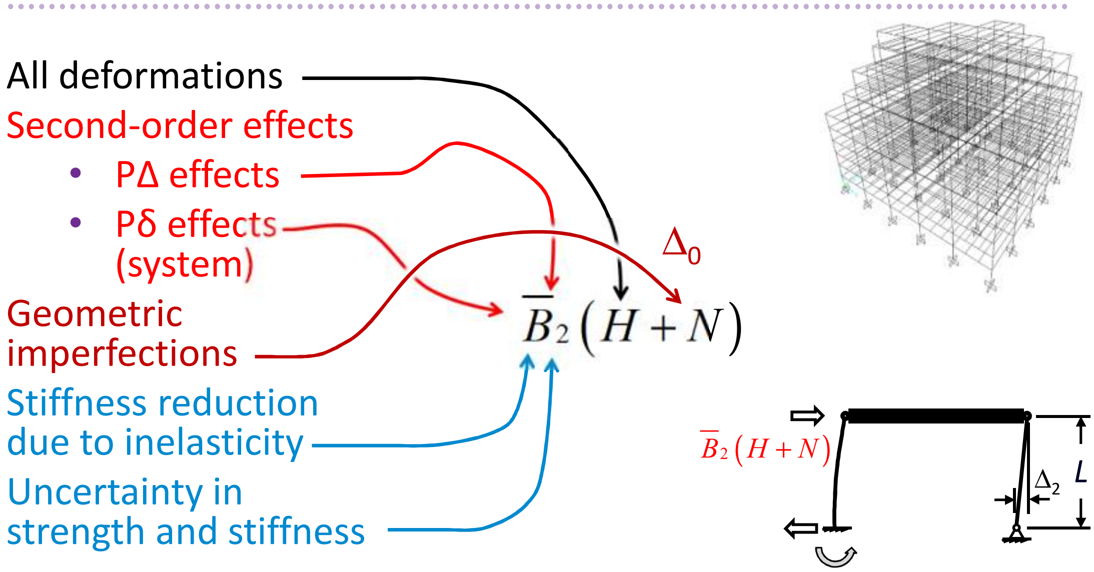

Equilibrium @ (Δ=0)→ Equilibrium @ ($\Delta_2$)

$$H + \bar{H}_{P\Delta} = \bar{B}_2(H + N)$$

First-order and second-order effects (including stiffness reduction) | Amplified first-order analysis (including imperfection loads)

111

## Equilibrium in the deformed condition : amplifier method

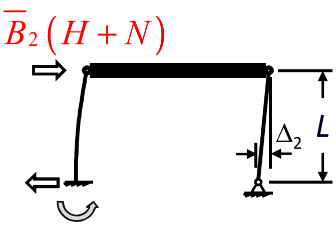



112

## Unpacking the amplifier

$\bar{B}_2(H+N)$

- $H$ First-order effects
- $N$ Initial imperfections




$$\bar{B}_2 = B_2 B_3$$

$$B_2 = \frac{1}{1-\frac{P_{story}\Delta_H}{R_M HL}}$$

Second-order effects
$P\delta$ and $P\Delta$


$$B_3 = \frac{0.8\tau_b}{1-[1-0.8\tau_b]B_2}$$

Stiffness-reduction effects

$$B_3 = \frac{4}{5-B_2}$$

NASCC: THE STEEL CONFERENCE

113

113

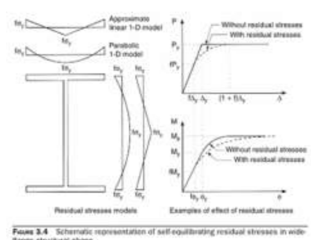
## Indirect analysis method

Parameter $\tau_b$

- Based on $\alpha P_r/P_{ns}$ (=$P_u/P_y$)
- Calculated member by member

Factor $B_3$

- Calculated story by story
  - Stiffness reduction of one member affects all members
- Applied story by story
  - Largest value can be applied to entire structure



NASCC: THE STEEL CONFERENCE

114

114

## 2 ways to implement

| | Pre-design | Loads | Analysis/ Design | Check | Iteration |
|---|---|---|---|---|---|
| Amplified 1st order analysis = 2nd order Analysis | Estimate $B_2$ and $B_3$ based on drift limit | Apply $B_2$ and $B_3$ to lateral forces | 1st order analysis | $\alpha P_r/P_{ns}$ ≤0.5 | If required |
| Explicit 2nd order Analysis | Estimate $B_2$ and $B_3$ based on drift limit | Apply $B_3$ to lateral forces | 2nd order analysis | $\alpha P_r/P_{ns}$ ≤0.5 | If required |

NASCC: THE STEEL CONFERENCE

115

115

## Indirect analysis method

Two ways to implement

- Amplified 1st-order analysis
  - Second-order effects approximated by $B_2$
  - Stiffness reduction using $B_3$ based on $B_2$
  - Both can be determined based on drift limit in advance of design

$$\bar{B}_2(H+N) = B_2B_3(H+N)$$

| $\alpha P_r/P_{ns}$ | ≤0.5 | 0.55 | 0.6 | 0.65 | 0.7 |
|---|---|---|---|---|---|
| $\tau_b$ | 1 | 0.99 | 0.96 | 0.91 | 0.84 |
| $B_2$ | | | $B_2B_3$ | | $B_2$ l |
| 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| 1.05 | 1.06 | 1.06 | 1.07 | 1.07 | 1.08 |
| 1.10 | 1.13 | 1.13 | 1.13 | 1.14 | 1.16 |
| 1.15 | 1.19 | 1.20 | 1.20 | 1.22 | 1.24 |
| 1.20 | 1.26 | 1.27 | 1.28 | 1.30 | 1.33 |
| 1.25 | 1.33 | 1.34 | 1.35 | 1.38 | 1.42 |
| 1.30 | 1.41 | 1.41 | 1.43 | 1.46 | 1.52 |
| 1.35 | 1.48 | 1.49 | 1.51 | 1.55 | 1.63 |
| 1.40 | 1.56 | 1.56 | 1.59 | 1.65 | 1.74 |
| 1.45 | 1.63 | 1.64 | 1.68 | 1.74 | 1.86 |
| 1.50 | 1.71 | 1.73 | 1.77 | 1.84 | 1.98 |

NASCC: THE STEEL CONFERENCE

116

116

## Indirect analysis method

Two ways to implement

$$\bar{B}_2(H+N) = \text{SOA}[B_3(H+N)]$$

- Explicit 2nd-order analysis
  - 2nd-order effects using software
  - Stiffness reduction using $B_3$
    - based on approximate $B_2$
      - » based on drift limit
        - in advance of design
    - Or $\Delta_2/\Delta_1$ (or $H_2/H_1$) from analysis

| $\alpha P_r/P_{ns}$ | ≤0.5 | 0.55 | 0.6 | 0.65 | 0.7 |
|---|---|---|---|---|---|
| $\tau_b$ | 1 | 0.99 | 0.96 | 0.91 | 0.84 |
| $B_2$ | | | $B_3$ | | B |
| 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| 1.05 | 1.01 | 1.01 | 1.02 | 1.02 | 1.03 |
| 1.10 | 1.03 | 1.03 | 1.03 | 1.04 | 1.05 |
| 1.15 | 1.04 | 1.04 | 1.05 | 1.06 | 1.08 |
| 1.20 | 1.05 | 1.06 | 1.06 | 1.08 | 1.11 |
| 1.25 | 1.07 | 1.07 | 1.08 | 1.10 | 1.14 |
| 1.30 | 1.08 | 1.09 | 1.10 | 1.13 | 1.17 |
| 1.35 | 1.10 | 1.10 | 1.12 | 1.15 | 1.21 |
| 1.40 | 1.11 | 1.12 | 1.14 | 1.18 | 1.24 |
| 1.45 | 1.13 | 1.13 | 1.16 | 1.20 | 1.28 |
| 1.50 | 1.14 | 1.15 | 1.18 | 1.23 | 1.32 |

NASCC: THE STEEL CONFERENCE

117

117

## Advantages and disadvantages: IAM and FOM


| | Advantages of IAM | Disadvantages of IAM |
|---|---|---|
| FOM | • **Lower forces**<br>• $\alpha P_r/P_{ns} \leq 0.7$ for moment-frame columns<br>• $B_2 \leq 2.0$<br>• **No additional lateral loads**<br>• $B_2B_3$ indicates stability | • **Second-order analysis**<br>*(But FOM is limited to $B_2 \leq 1.5$….)* |

NASCC: THE STEEL CONFERENCE

118

118

## Advantages and disadvantages: IAM and ELM


| | Advantages of IAM | Disadvantages of IAM |
|---|---|---|
| ELM | • Lower demand-to-capacity ratios (moment frames)<br>• ***K*=1**<br>• $B_2 \leq 2.0$<br>• **Provides appropriate design forces for connections & beams**<br>• $B_3$ indicates stability | • $\boldsymbol{B_3}$ amplifier<br>• $\alpha P_r/P_{ns} \leq 0.7$ for moment-frame columns |

NASCC: THE STEEL CONFERENCE

119

119

## Advantages and disadvantages: IAM and DM


| | Advantages of IAM | Disadvantages of IAM |
|---|---|---|
| DM | • **Single calculation of $\boldsymbol{B_3}$**<br>• **One model**<br>• $B_3$ is an indicator of stability | • **Potentially higher forces (largest $\boldsymbol{\tau_b}$)**<br>• Limited to<br>o **Vertical columns**<br>o $B_2 \leq 2.0$<br>o $\alpha P_r/P_{ns} \leq 0.7$ for moment-frame columns |

NASCC: THE STEEL CONFERENCE

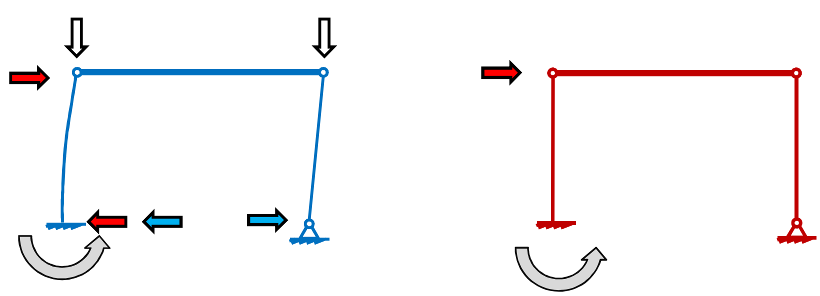
120

120


121

## Second-order analysis

Let's set aside ***N*** & $\boldsymbol{B_3}$ and focus on $\boldsymbol{B_2}$



$HL + \Sigma P\Delta_2$ = $B_2 HL$

$$B_2 = \frac{HL + P_{story}\Delta_2}{HL} = 1 + \frac{P_{story}\Delta_2}{HL}$$

NASCC: THE STEEL CONFERENCE

122

122

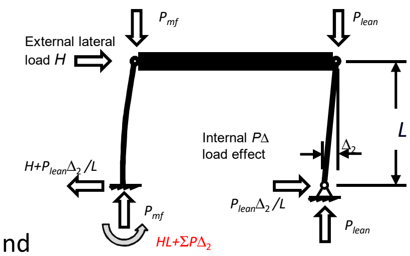
## Second-order analysis

$$B_2 = 1 + \frac{P_{story}\Delta_2}{HL}$$

Equilibrium at $\Delta_2$

$$B_2 = \frac{1}{1 - \frac{P_{story}\Delta_1}{R_M HL}}$$

Force amplifier with $P\Delta$ and $R_M$ stuffiness reduction




$$1 + \frac{P_{story}\Delta_2}{HL} = \frac{1}{1 - \frac{P_{story}\Delta_1}{R_M HL}}$$

$$\Delta_2 = \frac{B_2}{R_M}\Delta_1$$

NASCC: THE STEEL CONFERENCE

123

123

## Second-order analysis

$$1 + \frac{P_{story}\Delta_2}{HL} = \frac{1}{1 - \frac{P_{story}\Delta_1}{R_M HL}}$$

$$\frac{H}{\Delta_2} = R_M \frac{H}{\Delta_1} - \frac{P_{story}}{L}$$

- Effective stiffness
  - o 2nd order

$$\frac{H}{\Delta_1} = \frac{1}{R_M}\left(\frac{H}{\Delta_2} + \frac{P_{story}}{L}\right)$$

- Mechanical stiffness
  - o 1st order

NASCC: THE STEEL CONFERENCE

124

124

## Pδ stiffness reduction

$$B_2 = \frac{1}{1-\dfrac{P_{story}}{\dfrac{HL}{\Delta_1}-\left(\dfrac{12}{\pi^2}-1\right)P_{mf}}}$$

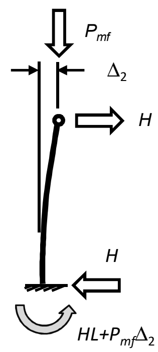
LeMessurier, 1977

$$\le \frac{1}{1-\dfrac{P_{story}\Delta_1}{\left(1-0.15\dfrac{P_{mf}}{P_{story}}\right)HL}}$$

AISC *Specification*

$$R_M = 1-\frac{P_{mf}\Delta_1}{HL}\left(\frac{12}{\pi^2}-1\right)$$

$$R_M = 1-0.216\theta\frac{P_{mf}}{P_{story}}$$




$$R_M = 1-0.15\frac{P_{mf}}{P_{story}}$$

AISC parameter $R_M$

Calibrated to θ=0.7

NASCC: THE STEEL CONFERENCE

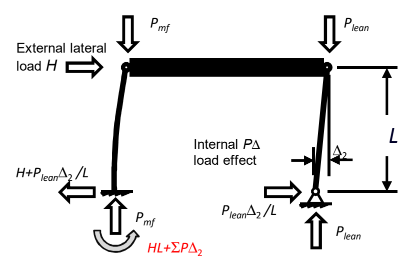
125

125

## Second-order analysis

$$B_2 = 1+\frac{P_{story}\Delta_2}{HL} \qquad B_2 = \frac{1}{1-\dfrac{P_{story}\Delta_1}{R_M HL}}$$

$$1+\frac{P_{story}\Delta_2}{HL} = \frac{1}{1-\dfrac{P_{story}\Delta_1}{R_M HL}}$$




$$\Delta_2 = \frac{B_2}{R_M}\Delta_1 \qquad R_M = 1-\frac{P_{mf}\Delta_1}{HL}\left(\frac{12}{\pi^2}-1\right)$$

**Based on LeMessurier, 1977**

$B_2$ with AISC $R_M$ ~ true $B_2/R_M$ for:
θ≤0.3 with $P_{mf}/P_{story}$ =1.0
θ≤0.5 with $P_{mf}/P_{story}$ =1/3

$$R_M = 1-0.15\frac{P_{mf}}{P_{story}}$$

NASCC: THE STEEL CONFERENCE

126

126

## Second-order analysis

$$\frac{H}{\Delta_2} = R_M \frac{H}{\Delta_1} - \frac{P_{story}}{L} = \frac{H}{\Delta_1} - \frac{P_{story} + \left(12/\pi^2 - 1\right) P_{mf}}{L}$$

$$\frac{H}{\Delta_1} = \frac{1}{R_M}\left(\frac{H}{\Delta_2} + \frac{P_{story}}{L}\right) = \frac{H}{\Delta_2} + \frac{P_{story} + \left(12/\pi^2 - 1\right) P_{mf}}{L}$$

$$\frac{P_{story} + \left(12/\pi^2 - 1\right) P_{mf}}{L} = \frac{P_{lean} + 12/\pi^2 \, P_{mf}}{L}$$

| $P\Delta$ effect | | $P\delta$ effect |
|---|---|---|
| $P_{lean}\Delta_2/L$ | $P_{mf}\Delta_2/L$ | $(12\pi^2-1)P_{mf}\Delta_2/L$ |
| Leaning columns | Moment-frame columns | |

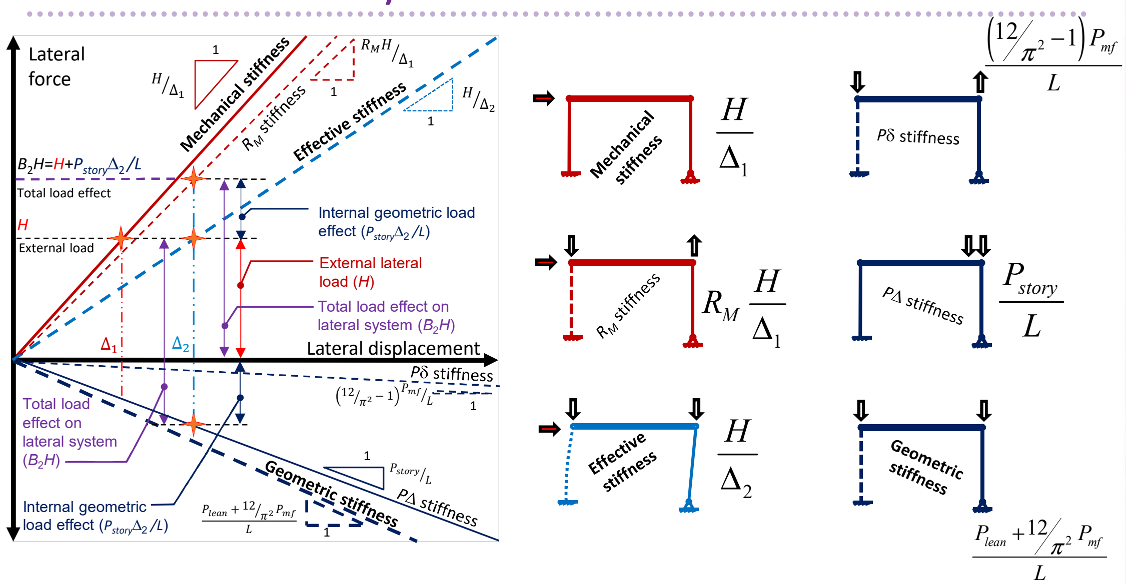
- Effective stiffness
  - o 2nd order
- Mechanical stiffness
  - o 1st order
- Geometric stiffness
  - o $P\Delta$ and $P\delta$

NASCC: THE STEEL CONFERENCE

127

127

## Second-order analysis




NASCC: THE STEEL CONFERENCE

128

128

## Determine $B_2$ in advance of design

$$B_2 = 1 + \frac{P_{story}\Delta_2}{HL} \qquad \Delta_2 = \frac{\Delta_{all}}{C_d} \qquad B_2 = 1 + \left[\frac{P_{story}}{C_d H}\right]\left[\frac{\Delta_{all}}{L}\right]$$

$C_d$=1.0 for wind

$H$ is load corresponding to $\Delta_{all}/C_d$
(This is a stiffness term!)

| $\Delta_{all}/L$ | $\Delta_{all}$ \ $P_{story}/C_d H$ | 2 | 4 | 5 | 6.7 | 8 | 10 | 15 | 20 | 25 | 33 | 40 | 50 | 60 | 80 | 100 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Values of $B_2$ | | | | | | | | | | | | | | |
| 0.0025 | L/400 | 1.01 | 1.01 | 1.01 | 1.02 | 1.02 | **1.03** | 1.04 | 1.05 | 1.06 | 1.08 | 1.10 | 1.13 | 1.15 | 1.20 | 1.25 |
| 0.0050 | L/200 | 1.01 | 1.02 | 1.03 | 1.03 | 1.04 | **1.05** | 1.08 | 1.10 | 1.13 | 1.17 | 1.20 | 1.25 | 1.30 | 1.40 | 1.50 |
| 0.0100 | L/100 | 1.02 | 1.04 | 1.05 | 1.07 | 1.08 | **1.10** | 1.15 | 1.20 | 1.25 | 1.33 | 1.40 | 1.50 | 1.60 | 1.80 | 2.00 |
| 0.0150 | L/67 | 1.03 | 1.06 | 1.08 | 1.10 | 1.12 | **1.15** | 1.23 | 1.30 | 1.38 | 1.50 | 1.60 | 1.75 | 1.90 | 2.20 | 2.50 |
| 0.0200 | L/50 | 1.04 | 1.08 | 1.10 | 1.13 | 1.16 | **1.20** | 1.30 | 1.40 | 1.50 | 1.67 | 1.80 | 2.00 | 2.20 | 2.60 | 3.00 |
| **0.0250** | **L/40** | **1.05** | **1.10** | **1.13** | **1.17** | **1.20** | **1.25** | 1.38 | 1.50 | 1.63 | 1.83 | 2.00 | 2.25 | 2.50 | 3.00 | 3.50 |

NASCC: THE STEEL CONFERENCE

129

129

## Determine $B_2$ and $B_3$ in advance of design

$$B_2 = 1 + \frac{P_{story}\Delta_2}{HL}$$

$$B_2 = 1 + \frac{P_{story}\Delta_{all}}{C_d HL}$$

$H$ is load corresponding to $\Delta_{all}/C_d$

$$B_3 = \frac{0.8\tau_b}{1 - [1 - 0.8\tau_b]B_2}$$

| $\frac{P_{story}\Delta_{all}}{C_d HL}$ | $P_u/P_y$ | ≤0.5 | 0.55 | 0.60 | 0.65 | 0.70 |
|---|---|---|---|---|---|---|
| | $\tau_b$ | 1.00 | 0.99 | 0.96 | 0.91 | 0.84 |
| | $B_2$ | $B_3$ | | | | |
| 0.025 | 1.03 | **1.01** | 1.01 | 1.01 | 1.01 | 1.01 |
| 0.050 | 1.05 | **1.01** | 1.01 | 1.02 | 1.02 | 1.03 |
| 0.075 | 1.08 | **1.02** | 1.02 | 1.02 | 1.03 | 1.04 |
| 0.100 | 1.10 | **1.03** | 1.03 | 1.03 | 1.04 | 1.05 |
| 0.125 | 1.13 | **1.03** | 1.03 | 1.04 | 1.05 | 1.06 |
| 0.150 | 1.15 | **1.04** | 1.04 | 1.05 | 1.06 | 1.08 |
| 0.175 | 1.18 | **1.05** | 1.05 | 1.06 | 1.07 | 1.09 |
| 0.200 | 1.20 | **1.05** | 1.06 | 1.06 | 1.08 | 1.11 |
| 0.225 | 1.23 | **1.06** | 1.06 | 1.07 | 1.09 | 1.12 |
| 0.250 | 1.25 | **1.07** | 1.07 | 1.08 | 1.10 | 1.14 |
| 0.275 | 1.28 | 1.07 | 1.08 | 1.09 | 1.11 | 1.16 |
| 0.300 | 1.30 | 1.08 | 1.09 | 1.10 | 1.13 | 1.17 |
| 0.325 | 1.33 | 1.09 | 1.09 | 1.11 | 1.14 | 1.19 |

NASCC: THE STEEL CONFERENCE

130

130

## Summary

Mechanical, effective, and geometric stiffness are related

- Geometric stiffness includes $P\Delta$ and $P\delta$ effects
- Effective stiffness is true stiffness in presence of vertical loads
- Effective = Mechanical – Geometric

Stability is based on equilibrium in the deformed condition

- Drift limits correspond to the deformed condition
- Stability amplifiers can be determined from the drift limit

NASCC: THE STEEL CONFERENCE

131

131


132

## Example design

8-story moment frame

Indirect analysis method

- Compare results with Direct Analysis

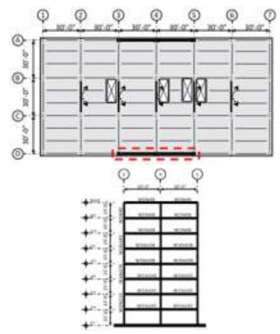
Wind design (no seismic)

Size members for drift

Check strength/stability

Factors determined based on drift limit

– (Appendix B of *Indirect Analysis Method,* AISC Engineering Journal, Quarter 2, 2020)

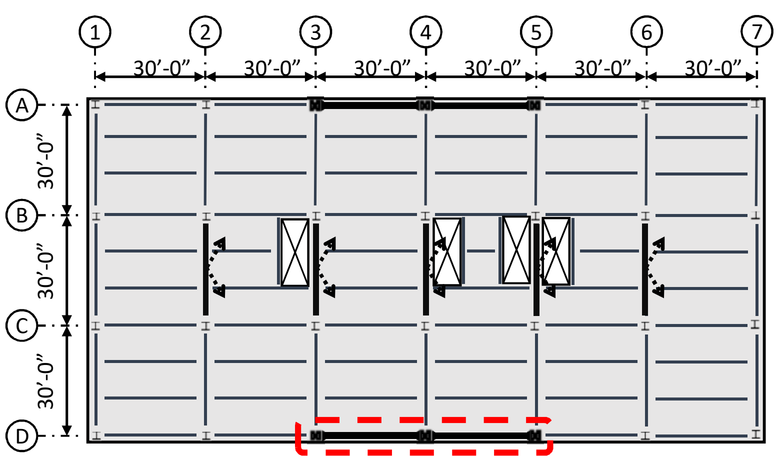


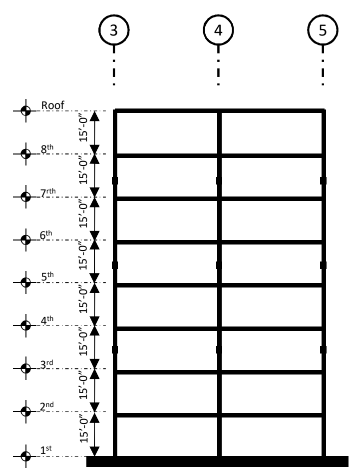

NASCC: THE STEEL CONFERENCE

133

133

## Plan and elevation




NASCC: THE STEEL CONFERENCE

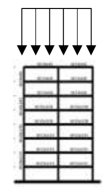
134

134

## Vertical loads

| General | | | | | |
|---|---|---|---|---|---|
| | Story Height | Dead Load | Live Load | $R_M = 1 - 0.15 \, P_{mf}/P_{story}$ | |
| *Level* | *L* (in.) | *Dead Load* (kip) | *Live Load* (kip) | $P_{mf}/P_{story}$ | $R_M$ |
| 8 | 180 | 2000 | 0 | 0.275 | 0.959 |
| 7 | 180 | 2000 | 1600 | 0.275 | 0.959 |
| 6 | 180 | 2000 | 1600 | 0.275 | 0.959 |
| 5 | 180 | 2000 | 1600 | 0.275 | 0.959 |
| 4 | 180 | 2000 | 1600 | 0.275 | 0.959 |
| 3 | 180 | 2000 | 1600 | 0.275 | 0.959 |
| 2 | 180 | 2000 | 1600 | 0.275 | 0.959 |
| 1 | 180 | 2000 | 1600 | 0.275 | 0.959 |

All pre-design information: no preliminary member design



NASCC: THE STEEL CONFERENCE

135

135

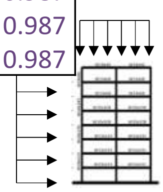
## Drift design

| Drift | | | L/400 | | | $\frac{H}{\Delta_1}$ | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | 1.0DL+0.25LL | | | | | $R_M = 1 - \frac{P_{mf}\Delta_1}{HL}\left(\frac{12}{\pi^2} - 1\right)$ | |
| *Level* | $H_{service}$ (kip) | $\Delta_{allowable}$ (in.) | $\frac{H}{\Delta_2}$ | $P_{story}$ (kip) | $K_{required}$ (kip/in) | $\Delta_1$ (in.) | $B_2$ | $R_M$ |
| 8 | 20.0 | 0.450 | | 2,000 | 56 | 0.357 | 1.26 | 0.988 |
| 7 | 40.0 | 0.450 | | 4,400 | 114 | 0.350 | 1.29 | 0.987 |
| 6 | 60.0 | 0.450 | | 6,800 | 173 | 0.347 | 1.30 | 0.987 |
| 5 | 80.0 | 0.450 | | 9,200 | 231 | 0.346 | 1.30 | 0.987 |
| 4 | 100.0 | 0.450 | | 11,600 | 289 | 0.345 | 1.30 | 0.987 |
| 3 | 120.0 | 0.450 | | 14,000 | 348 | 0.345 | 1.30 | 0.987 |
| 2 | 140.0 | 0.450 | | 16,400 | 406 | 0.345 | 1.31 | 0.987 |
| 1 | 160.0 | 0.450 | | 18,800 | 464 | 0.344 | 1.31 | 0.987 |

All pre-design information:
no preliminary member design

Technically correct Used in paper

$$\frac{H}{\Delta_1} \geq \frac{1}{R_M}\left(\frac{H}{\Delta_2} + \frac{\sum P}{L}\right) \approx \frac{H}{\Delta_2} + \frac{\sum P}{R_M L}$$



NASCC: THE STEEL CONFERENCE

136

136

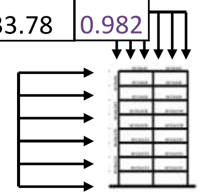
## Strength-design amplifiers

Strength — $\frac{H}{\Delta_1}$ From service-level drift limit — 1.2DL+0.5LL — $R_M = 1 - \frac{P_{mf}\Delta_1}{HL}\left(\frac{12}{\pi^2} - 1\right)$

| Level | H (kip) | F | K=$K_{required}$ (kip/in) | $\Delta_1$ (in.) | $P_{story}$ (kip) | $B_2$ | $B_3$ | $B_3F$ (kip) | $R_M$ |
|---|---|---|---|---|---|---|---|---|---|
| 8 | 30 | 30.0 | 56 | 0.535 | 2,400 | 1.33 | 1.09 | 33.78 | 0.986 |
| 7 | 60 | 30.0 | 114 | 0.525 | 5,600 | 1.40 | 1.11 | 33.78 | 0.984 |
| 6 | 90 | 30.0 | 173 | 0.521 | 8,800 | 1.42 | 1.12 | 33.78 | 0.983 |
| 5 | 120 | 30.0 | 231 | 0.519 | 12,000 | 1.43 | 1.12 | 33.78 | 0.983 |
| 4 | 150 | 30.0 | 289 | 0.518 | 15,200 | 1.44 | 1.12 | 33.78 | 0.983 |
| 3 | 180 | 30.0 | 348 | 0.518 | 18,400 | 1.44 | 1.12 | 33.78 | 0.983 |
| 2 | 210 | 30.0 | 406 | 0.517 | 21,600 | 1.45 | 1.13 | 33.78 | 0.982 |
| 1 | 240 | 30.0 | 464 | 0.517 | 24,800 | 1.45 | **1.13** | 33.78 | 0.982 |

All pre-design information: no preliminary member design

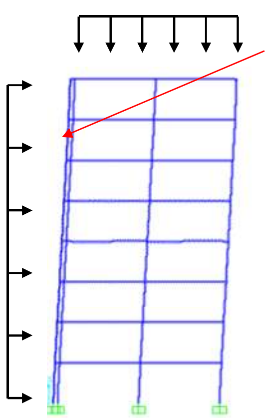
$$B_3 = \frac{0.8\tau_b}{1-\left[1-0.8\tau_b\right]B_2} = \frac{4}{5-B_2}$$



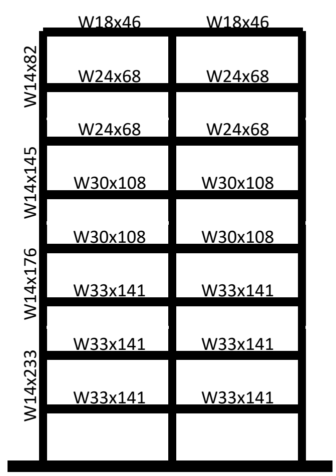
NASCC: THE STEEL CONFERENCE 137

137

## Design for drift




Second-order analysis
Iterate to optimize for drift




Sizes selected

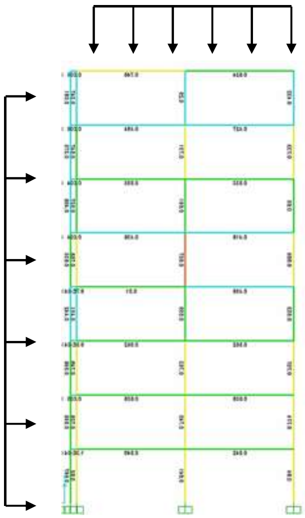
| Level | $\Delta_2/\Delta_{all}$ |
|---|---|
| 8 | 0.61 |
| 7 | 0.96 |
| 6 | 0.89 |
| 5 | 0.95 |
| 4 | 0.93 |
| 3 | 1.00 |
| 2 | 0.93 |
| 1 | 0.72 |

Drifts very close to the limit

NASCC: THE STEEL CONFERENCE 138

138

## Strength/Stability check



| $aP_r/P_{ns}$ | $\tau_b$ | $\Delta_1$ (in.) | $\Delta_2$ (in.) | $B'_2$ | $B'_3$ |
|---|---|---|---|---|---|
| 0.07 | 1.00 | 0.39 | 0.49 | 1.25 | 1.07 |
| 0.16 | 1.00 | 0.59 | 0.79 | 1.35 | 1.10 |
| 0.15 | 1.00 | 0.54 | 0.74 | 1.36 | 1.10 |
| 0.21 | 1.00 | 0.57 | 0.79 | 1.38 | 1.11 |
| 0.22 | 1.00 | 0.56 | 0.78 | 1.39 | 1.11 |
| 0.28 | 1.00 | 0.60 | 0.84 | 1.41 | 1.11 |
| 0.25 | 1.00 | 0.56 | 0.78 | 1.38 | 1.11 |
| 0.29 | 1.00 | 0.45 | 0.59 | 1.32 | 1.09 |

Second-order analysis
$B'_2 = \Delta_2/\Delta_1$
$B'_3$ calculated from $B'_2$
(for comparison only)

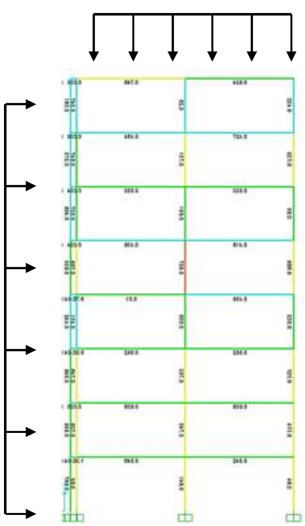
Correct stiffness
Correct $R_M$ via
proper $P\delta$
modeling

NASCC: THE STEEL CONFERENCE

139

139

## Strength/Stability check



| Beam check | | | Column check | | |
|---|---|---|---|---|---|
| IAM Beam (DCR) | DM Beam (DCR) | Ratio IAM/DM | IAM Column (DCR) | DM Column (DCR) | Ratio IAM/DM |
| 0.13 | 0.12 | 1.06 | 0.14 | 0.14 | 1.04 |
| 0.21 | 0.20 | 1.04 | 0.45 | 0.45 | 1.01 |
| 0.31 | 0.30 | 1.02 | 0.25 | 0.25 | 1.02 |
| 0.27 | 0.27 | 1.02 | 0.45 | 0.45 | 1.01 |
| 0.32 | 0.31 | 1.02 | 0.47 | 0.47 | 1.01 |
| 0.29 | 0.29 | 1.01 | 0.58 | 0.58 | 1.01 |
| 0.33 | 0.32 | 1.01 | 0.50 | 0.50 | 1.01 |
| 0.33 | 0.32 | 1.03 | 0.65 | 0.64 | 1.02 |

IAM: Indirect analysis method
DM: Direct analysis method

NASCC: THE STEEL CONFERENCE

140

140

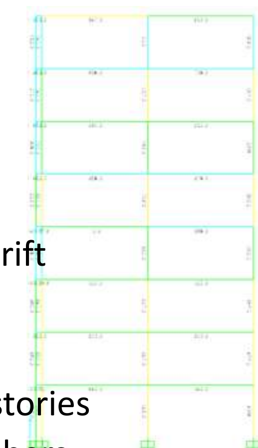
## Example summary

Indirect analysis method is easy to implement

- One model
- $K$=1

Indirect analysis method is conservative

- Factors determined based on drift limit
  - Could be re-calculated based on actual drift
- Specification $R_M$ is conservative
- Worst case $B_3$ applied all stories
  - Lower values could be applied at upper stories
- Conservatism does not result in larger members



NASCC: THE STEEL CONFERENCE

141

141

# Summary

NASCC:
THE STEEL CONFERENCE

142

## Summary

Design for stability is necessary

- Must address "Big Five" considerations, including:
  - 2nd-order effects
    - – $B_2$ amplifier
      - » Or $B_2$–based force (FOM)
    - – Explicit second-order analysis
  - Stiffness-reduction effects
    - – ELM: *K*
    - – DM: Reduced stiffness ($0.8\tau_b$)
    - – FOM: 0.8–based force; axial-force limit

NASCC: THE STEEL CONFERENCE 143

143

## Summary

Indirect Analysis Method: useful option for moment frames

- $K$=1
- One model
- $B_3$ addresses stiffness-reduction effects
  - $0.8\tau_b$
  - Function of $B_2$

NASCC: THE STEEL CONFERENCE 144

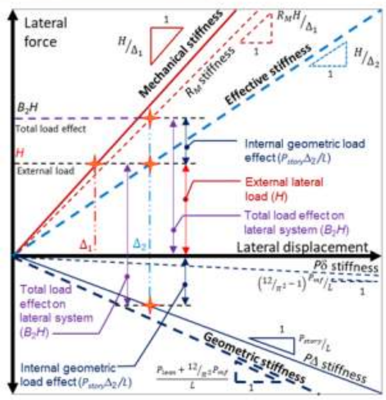
144

## Summary

Second-order effects part of design for stability
- included in the analysis
- or addressed via amplifiers

Mechanical, effective, and geometric stiffness
- Geometric stiffness includes PΔ and Pδ
- Effective stiffness is true stiffness
- Effective = Mechanical – Geometric

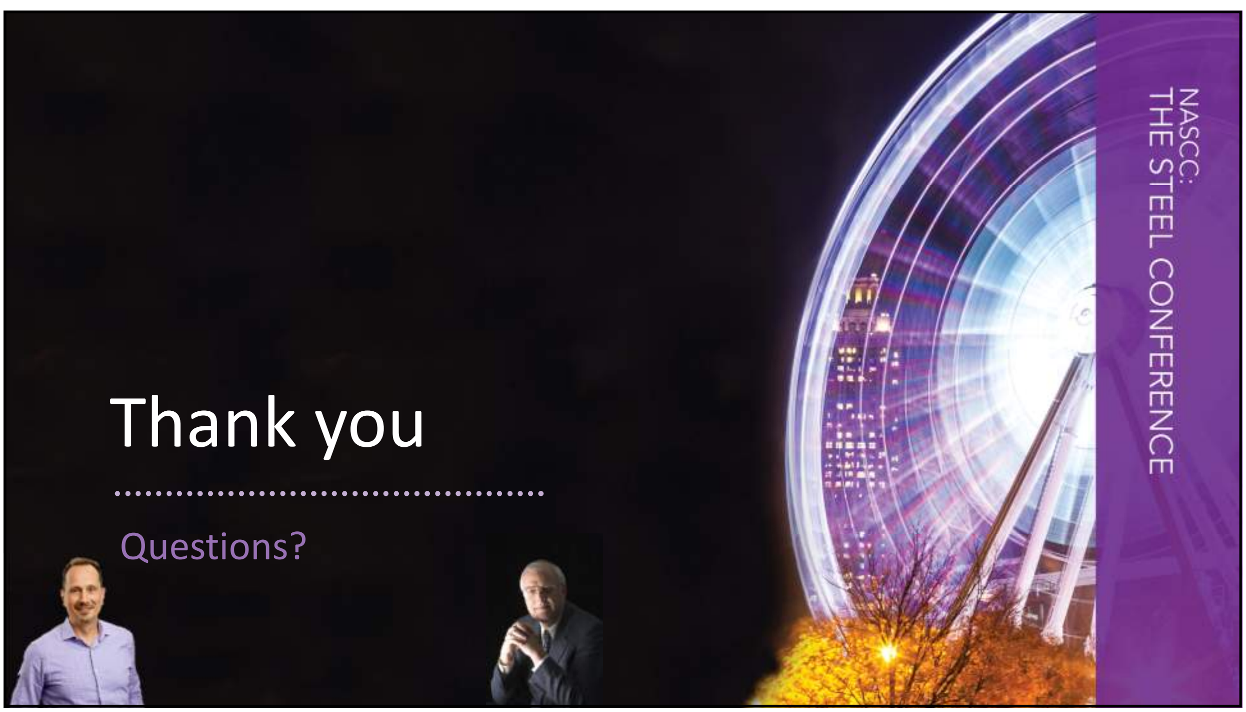
Equilibrium in the deformed condition
- Drift limits correspond to the deformed condition
- Amplifiers can be determined from drift limit




NASCC: THE STEEL CONFERENCE

145

145




146


AMERICAN INSTITUTE OF STEEL CONSTRUCTION
AISC
FOUNDED 1921
NASCC: THE
VIRTUAL
STEEL
CONFERENCE

147