Ph.D. Defense

Design Philosophy and Parametric Collapse Performance of Low-Ductility Concentrically Braced Frames With Reserve Capacity

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July 16, 2019

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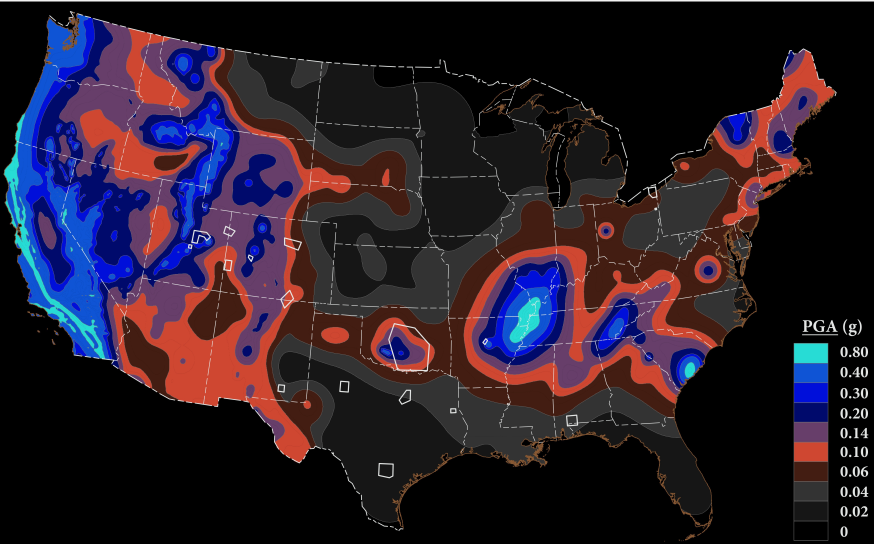
Agenda

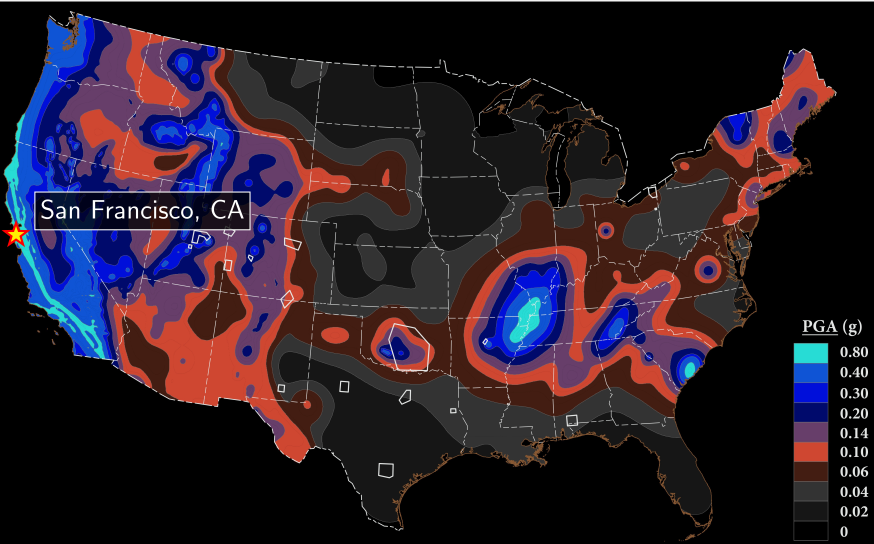
- Introduction
- Experimental Program
- Parametric Study
- Numerical Modeling
- Simulation Results
- Conclusions

Introduction



2 % Prob. of Exceedance in 50 Years (USGS, 2014)

Introduction



2 % Prob. of Exceedance in 50 Years (USGS, 2014)

Introduction

Ductility-Based Design Framework

- For a given intensity of ground shaking, buildings with substantial ductility can suffer considerably more damage without collapse than buildings with limited ductility.
- Therefore, systems with greater ductility can be designed for smaller seismic forces to achieve an equal probability of acceptable performance under a design-level earthquake.

Introduction

Ductility-Based Design Framework

- Ductility corresponds to a parameter called the response modification coefficient, R.
- Each category of seismic force-resisting system (SFRS) is assigned an R-factor based upon general toughness and expected ductility.
- The R-factor adjusts the magnitude of seismic design forces:

$$V = \frac{S_{DS} W}{R}$$

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Seismic Design Base Shear →

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Spectral Acc. Parameter ↙ S_{DS}

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Ductility-Based Design Framework

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Spectral Acc. Parameter

Seismic Design Base Shear → $$V = \frac{S_{DS} W}{R}$$ ← Seismic Weight

Ductility-Based Design Framework

- Ductility corresponds to a parameter called the response modification coefficient, *R*.
- Each category of seismic force-resisting system (SFRS) is assigned an *R*-factor based upon general toughness and expected ductility.
- The *R*-factor adjusts the magnitude of seismic design forces:

Spectral Acc. Parameter

Seismic Design Base Shear → $V = \dfrac{S_{DS} W}{R}$ ← Seismic Weight

← *R*-factor

Introduction

Ductility-Based Design Framework

- The R-factor is inversely proportional to the seismic design base shear.
- Thus, systems with more extensive ductility are assigned larger R-factors, which correspond to larger reductions in seismic design forces.
- This allows engineers to select the seismic force-resisting system (SFRS) that provides a balance between magnitude of design forces and cost of ductility detailing.

Moderate-Seismic Considerations

- The maximum expected earthquake magnitude in Massachusetts is similar to that in California.
- However, the return periods are substantially longer in MA (5,000 yrs) than in CA (100 yrs).
- Thus, the cost to society of high-ductility design in Mass. is “considerably greater than the projected savings in damage and loss of life due to an earthquake” (Luft and Simpson, 1979).

Moderate-Seismic Considerations

- Life-safety must take precedence over economic considerations of structural damage.
- It is necessary to distinguish between magnitude of seismic forces and ductility capacity.
- In 1974, these design recommendations were incorporated into the first edition of the *Massachusetts State Building Code*.

Moderate-Seismic Challenges

- Acceptance of potential structural damage in exchange for reduced detailing requirements.
- For Concentrically Braced Frame (CBF) systems, this is problematic because:
    1. CBFs are relatively strong and stiff.
    2. High-ductility CBFs have $R = 6$ to 8.
    3. The inherent ductility of steel systems is defined as $R = 3$.

Standard CBF Design Options

- Special Concentrically Braced Frame (SCBF)
    - High-Ductility
    - $R = 6$
- Ordinary Concentrically Braced Frame (OCBF)
    - Moderate-Ductility
    - $R = 3.25$
- Steel Systems not Detailed for Seismic Resistance
    - Low-Ductility
    - $R = 3$

OCBF Versus $R = 3$

- The OCBF requirements have been minimalized, and relative to the $R = 3$ system:
  1. The R-factor provides a reduction in design forces of only 8 %.
  2. The ductility is not expected to result in substantially superior collapse performance.
- Thus, the $R = 3$ system has proven to be a cost-effective design solution in regions of moderate-seismic hazard.

Introduction

ATC 19 Report (ATC, 1995)

- There is no mathematical basis for the response modification (R) factors tabulated in modern seismic codes in the United States.
- A single value of R for all buildings of a given framing type, irrespective of plan and vertical geometry, cannot be justified.
- The values currently assigned to R for different framing systems will probably not result in uniform levels of risk for all buildings.

Introduction

Hines et al. (2009)

1. How much ductility, how much strength, and how much reserve capacity are actually required for a building to survive an MCE event in a moderate seismic region?
2. In particular, can an inherently low-ductility system such as a concentric chevron braced steel frame be designed to survive moderate seismic demand with a high level of confidence?

⋮

Introduction

Hines et al. (2009)

⋮

3. Should such systems even be evaluated according to the concepts of ductility and capacity design?
4. How can we articulate a design philosophy for low-ductility systems in moderate seismic regions that aims to ensure safety against collapse while allowing engineers as much freedom as possible to design creatively and economically?

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Experimental Program

Collaborative Research Team

University of Illinois at Urbana-Champaign

- Dr. Larry Fahnestock (PI)
- Josh Sizemore (RA, PhD student)

Tufts University & LeMessurier

- Dr. Eric Hines (Co-PI)
- Cameron Bradley (RA, PhD student)
- Jessalyn Nelson (RA, MS student)

École Polytechnique de Montréal

- Dr. Robert Tremblay (Co-PI)
- Thierry Beland (RA, PhD student)
- Ali Davaran (post-doctoral researcher)

Experimental Program

Research Objectives

- Improve the currently limited understanding of low-ductility braced frame failure mechanisms
- Contribute to existing literature and databases through structural tests and numerical simulations
- Help the profession assess and evaluate current seismic code requirements for low-to-moderate seismic regions

Experimental Program

Angle Component Tests

No. Tests	139
Year	2013–2014
Location	École Poly. Montréal
References	Nelson et al. (2014)
	Beland et al. (2014)
	Beland et al. (2015)
	Beland et al. (2019)

Purpose: Obtain strength and stiffness characteristics for a wide range of bolted-angle geometries.

Experimental Program

Brace Component Tests

No. Tests	6
Year	2013
Location	École Poly. Montréal
References	Davaran et al. (2014)

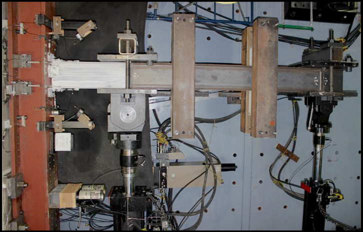
Purpose: Observe and identify the limit states of low-ductility slotted-HSS CBF connections.

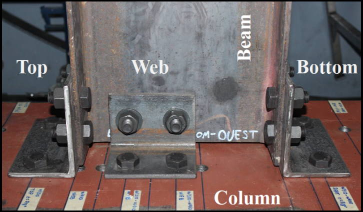
Experimental Program

Gravity Connection Tests

No. Tests	24
Year	2016–2017
Location	École Poly. Montréal
References	Beland et al. (2014) Beland et al. (2015) Beland et al. (2016) Béland et al. (2018)

Purpose: Obtain moment rotation characteristics for simple shear and semi-rigid beam-column gravity connections.





Experimental Program

Braced Frame Tests

No. Tests	2
Year	2014
Location	Lehigh University
References	Bradley (2016) Bradley et al. (2017) Sizemore (2017) Sizemore et al. (2017)

Purpose: Observe and identify types, locations, and hierarchy of post-elastic system mechanisms.

Experimental Program

Local Slenderness Effects

OCBF

R = 3

Experimental Program

Connection Behaviors


2014-09-26--15-34-38


2014-09-25--13-54-01

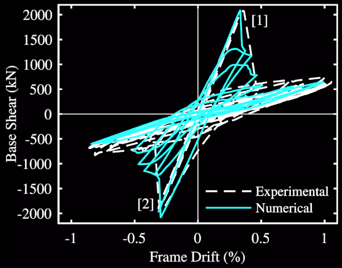

2014-09-25--15-07-32

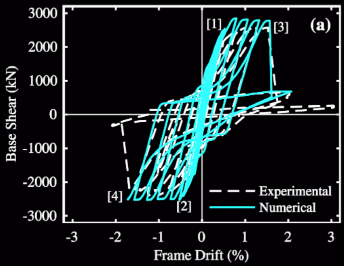
Experimental Program

Simulations

Observations from these experiments provided insights for understanding low-ductility frame behaviors from the component to system levels.

This allowed for the calibration and validation of numerical models that accurately predict the experimental results of the frame tests.





Plots from Sizemore et al. (2017)

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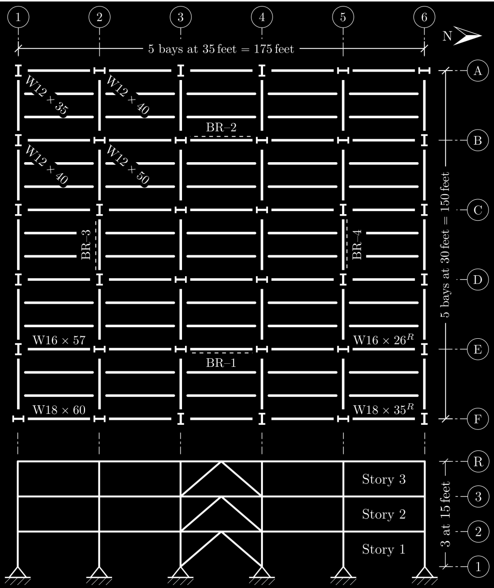
Parametric Study



Dead	
Slab	46[a]
Deck	2
Floor	2
Hung	10
Framing	5
Ponding	5
$\sum$	70
Live	
Partitions	15
Office	50
$\sum$	65
Roof	
Dead	30
Live	30
Snow	30
Other	
Facade	25

[a] 3.25–inch LWC on 3–inch steel deck.

Parametric Study

Design Parameters

		Variations			
Parameter	Description	Level 1	Level 2	Level 3	Level 4
R_P	Primary R-factor	3	4	5	–
R_R	Reserve R-factor	3	4	5	∞[a]
γ_b	Brace Slenderness[c]	0.64	0.80	0.96	–
γ_w	Weld Strength[b]	0.80	1.40	1.80	–
C_1	Capacity Design[d]	0	1	–	–

[a] Value of ∞ indicates systems designed without a reserve MRF.

[b] Coefficients of the brace local slenderness limit: $b/t \leq \gamma_b \sqrt{E/F_y}$.

[c] Ratios of modeled-to-nominal weld strengths: $R_{mw} = \gamma_w R_{nw}$.

[d] Values are binary for systems with and without a capacity-designed first story.

Design Parameters — R_P

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Parameter	Description	Level 1	Level 2	Level 3	Level 4
R_P	Primary R-factor	3	4	5	–
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Design Parameters — R_R

		Variations			
Parameter	Description	Level 1	Level 2	Level 3	Level 4
R_P	Primary R-factor	3	4	5	–
R_R	Reserve R-factor	3	4	5	∞[a]
γ_b	Brace Slenderness[c]	0.64	0.80	0.96	–
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C_1	Capacity Design[d]	0	1	–	–

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[c] Ratios of modeled-to-nominal weld strengths: $R_{mw} = \gamma_w R_{nw}$.

[d] Values are binary for systems with and without a capacity-designed first story.

Parametric Study

Design Parameters — γ_b

		Variations			
Parameter	Description	Level 1	Level 2	Level 3	Level 4
R_P	Primary R-factor	3	4	5	–
R_R	Reserve R-factor	3	4	5	∞[a]
γ_b	Brace Slenderness[c]	0.64	0.80	0.96	–
γ_w	Weld Strength[b]	0.80	1.40	1.80	–
C_1	Capacity Design[d]	0	1	–	–

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[c] Ratios of modeled-to-nominal weld strengths: $R_{mw} = \gamma_w R_{nw}$.

[d] Values are binary for systems with and without a capacity-designed first story.

Design Parameters — γ_w

		Variations			
Parameter	Description	Level 1	Level 2	Level 3	Level 4
R_P	Primary R-factor	3	4	5	–
R_R	Reserve R-factor	3	4	5	∞[a]
γ_b	Brace Slenderness[c]	0.64	0.80	0.96	–
γ_w	Weld Strength[b]	0.80	1.40	1.80	–
C_1	Capacity Design[d]	0	1	–	–

[a] Value of ∞ indicates systems designed without a reserve MRF.

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[d] Values are binary for systems with and without a capacity-designed first story.

Parametric Study

Design Parameters — C_1

		Variations			
Parameter	Description	Level 1	Level 2	Level 3	Level 4
R_P	Primary R-factor	3	4	5	–
R_R	Reserve R-factor	3	4	5	∞[a]
γ_b	Brace Slenderness[c]	0.64	0.80	0.96	–
γ_w	Weld Strength[b]	0.80	1.40	1.80	–
C_1	Capacity Design[d]	0	1	–	–

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Definition of SFRS Categories

SFRS	No.	Description
$R = 3$	1	Exempt from the *Seismic Provisions*
OCBF	1	Minimal seismic detailing and proportioning
CBDF-N	54	Parametric variations of modified OCBF provisions
CBDF-R	162	CBDF-N systems supplemented with reserve capacity
Total	218	

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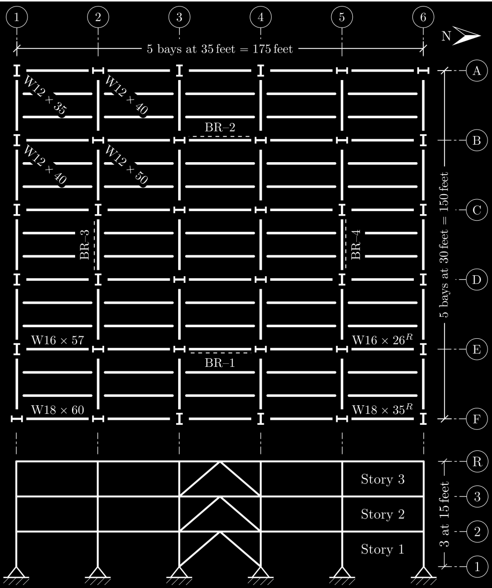
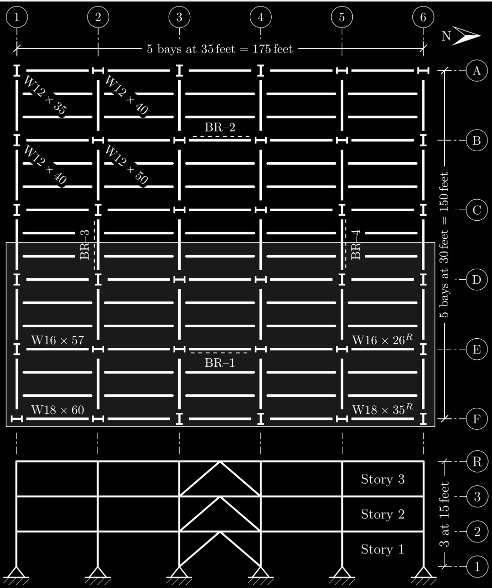
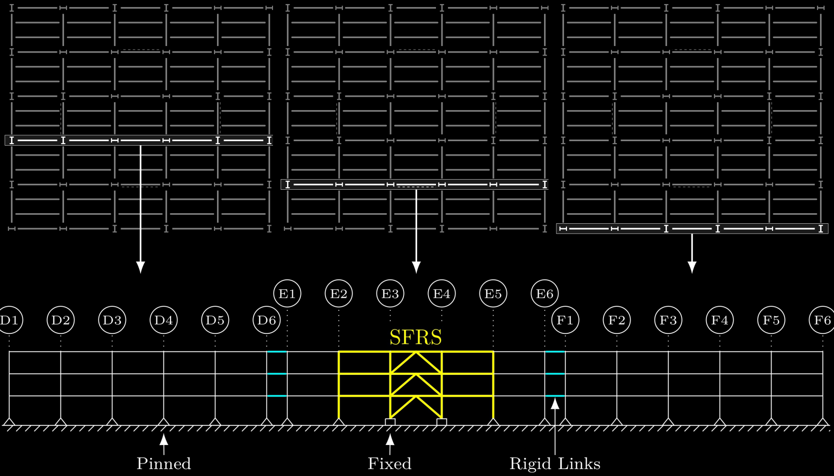
Numerical Modeling



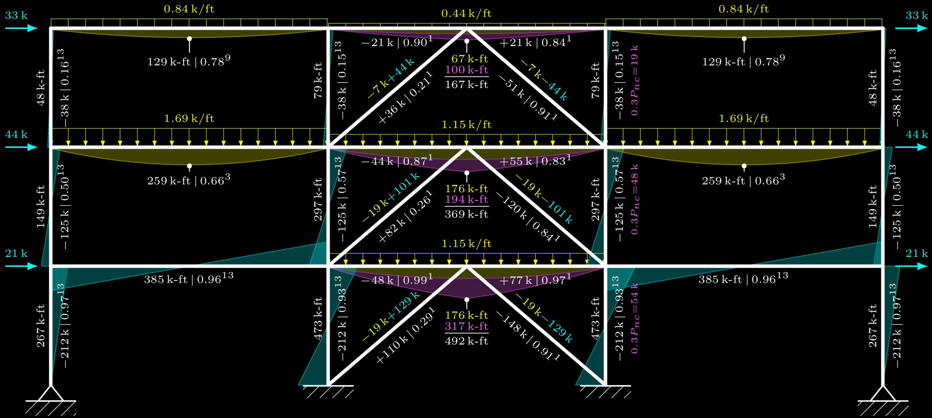
Numerical Modeling



Model Schematic

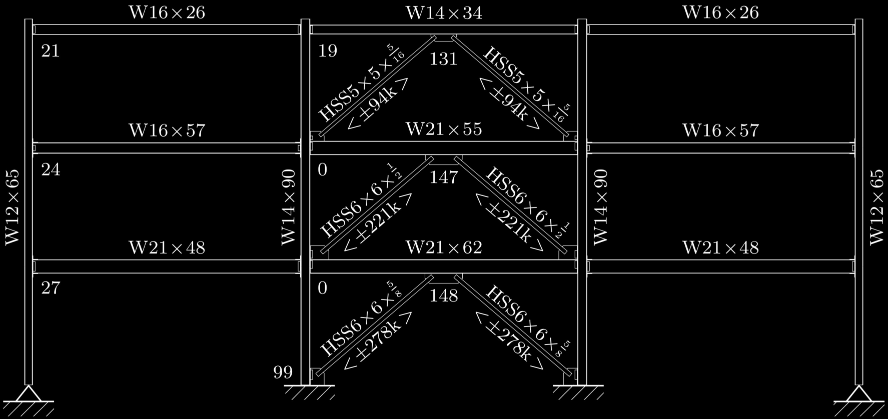


Governing Demands



Numerical Modeling

Resulting Design



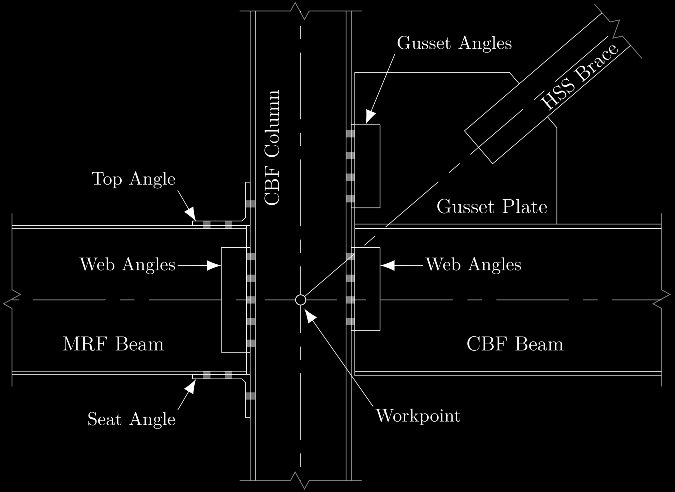
Numerical Modeling

Member Models

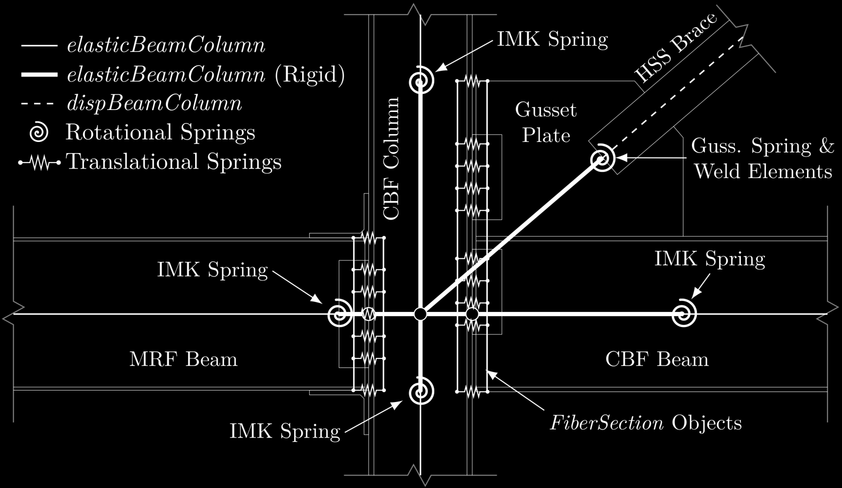
- Beams and strong-axis columns modeled as concentrated-plasticity members for comput ational efficiency.
- Braces and weak-axis columns were modeled as distributed-plasticitity members to capture P-δ, buckling, and geometric (i.e. local buckling) behaviors.
- Connections were modeled as zero-length fiber sections based upon the angle and connection tests.

Numerical Modeling

Connection Designs



Connection Models



Incremental Dynamic Analyses

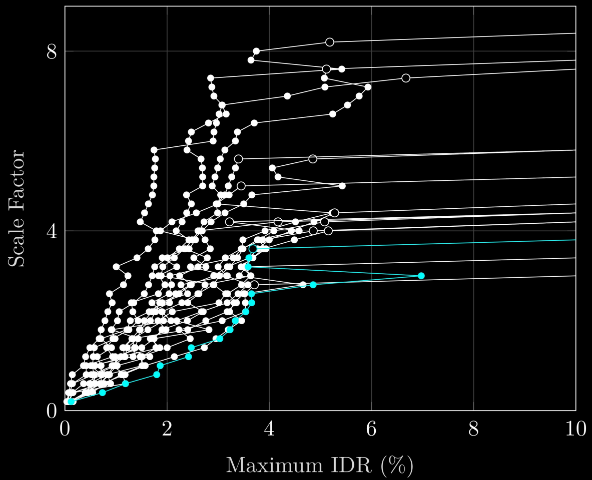
- Each prototype system was assessed using the incremental dynamic analysis (IDA) methodology.
- This involves simulating the response of a structure to a suite of ground motions while scaling the acceleration intensities until structural collapse is observed for each motion.
- The objective is to produce collapse fragility curves that can be combined with modeling uncertainties to assess collapse probability.

Numerical Modeling

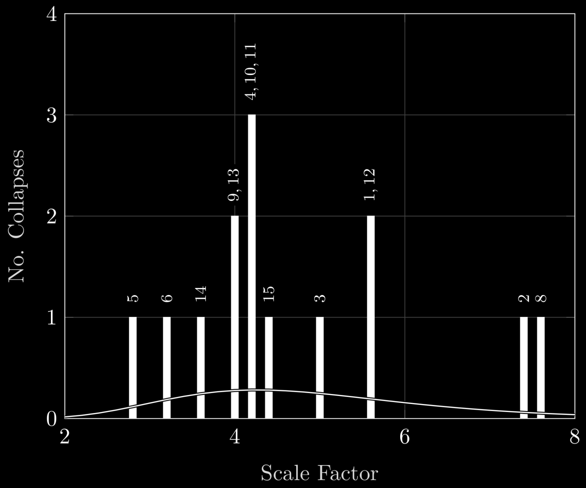
Incremental Dynamic Analyses

- A suite of 15 ground motions developed to be representative of Boston was used.
- Acceleration intensities were scaled in increments of 20 % of the unscaled motions.
- Structural collapse was defined as the point when either the roof drift ratio or any interstory drift ratio exceeded 10 %.
- Nonconvergent analyses were considered to have collapsed in the previous scale factor increment.

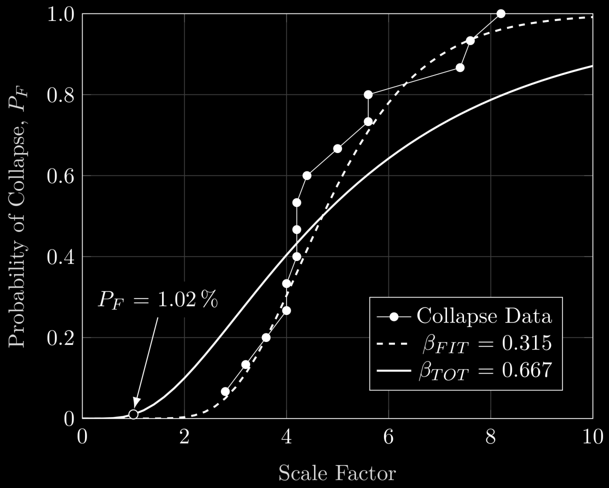
IDA Curves



Collapse Distribution



Fragility Curves



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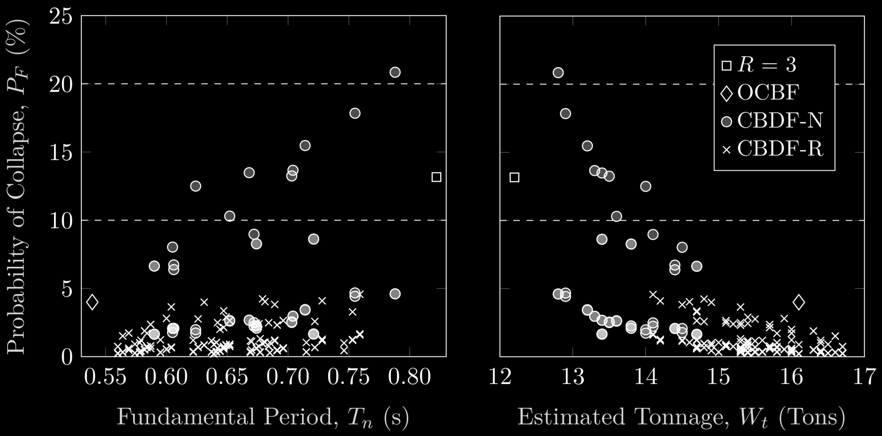
Simulation Results

Overall Results

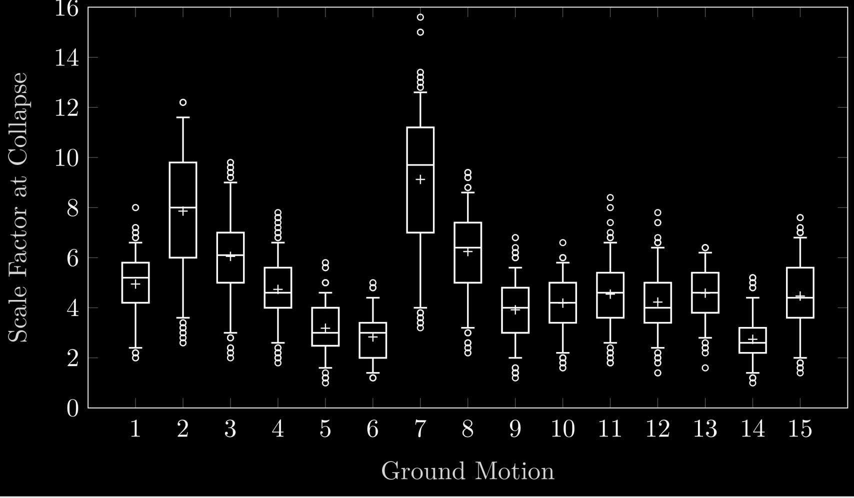
- A total of 218 variations of the prototype SFRS were analyzed with the IDA approach.
- A total of 83,145 individual ground motion simulations were conducted.
- Only 25 individual analyses failed to converge (approximately 0.03 %).
- The probability of collapse, P_F, at a scale factor of 1.0 ranged from 0.20 % to 20.8 %.

Overall Results



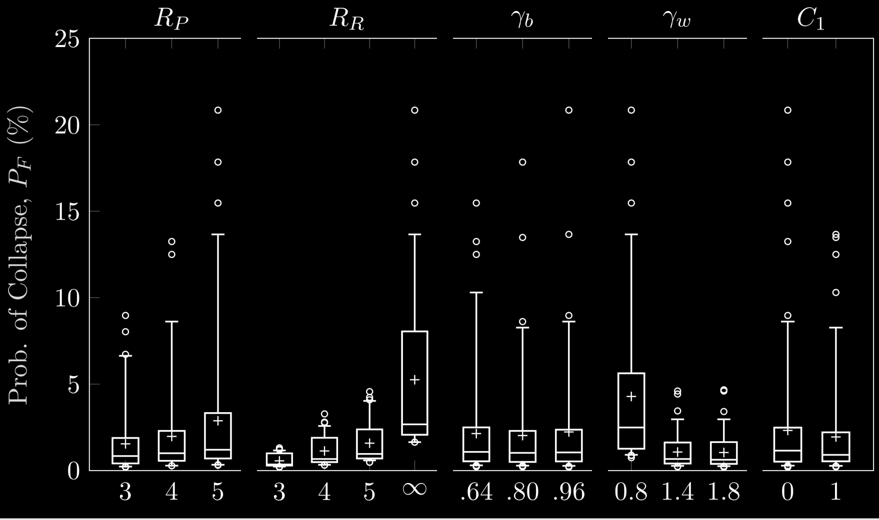
Simulation Results

Record-to-Record Variation



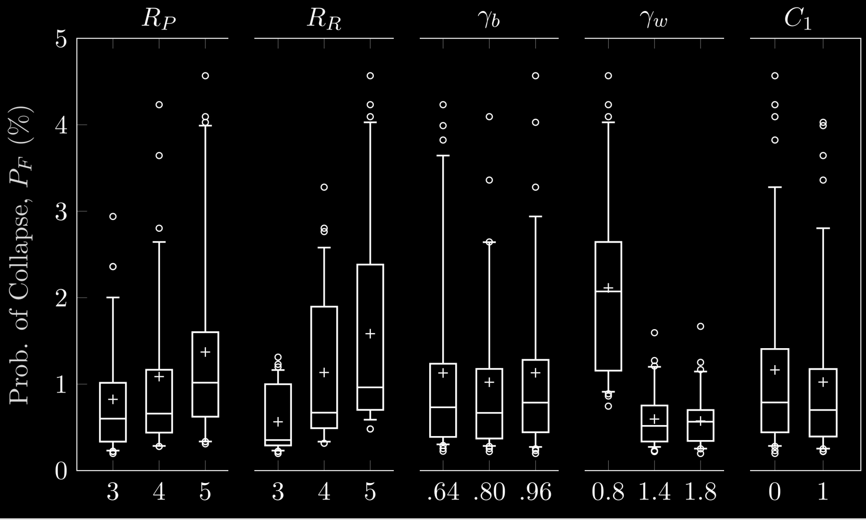
Simulation Results

CBDF Distributions



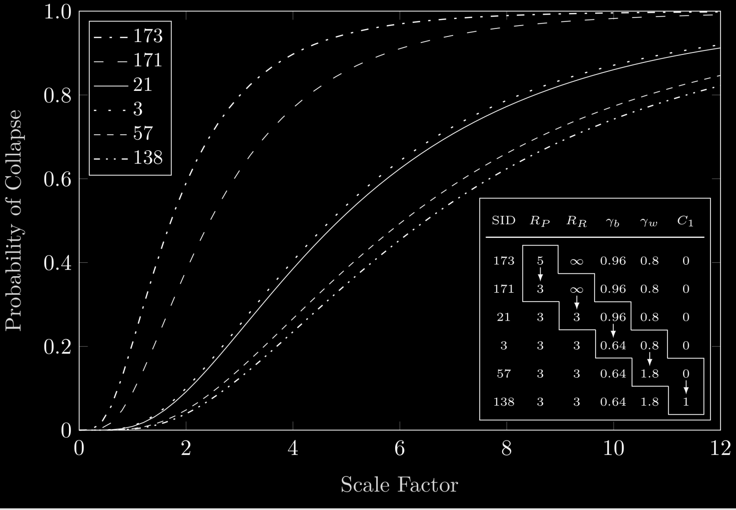
Simulation Results

CBDF-R Distributions



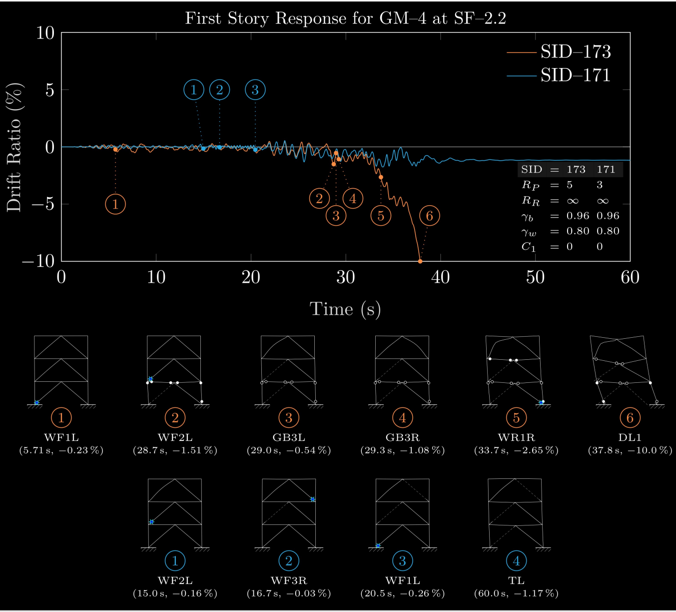
Simulation Results

Theoretical Collapse Progression



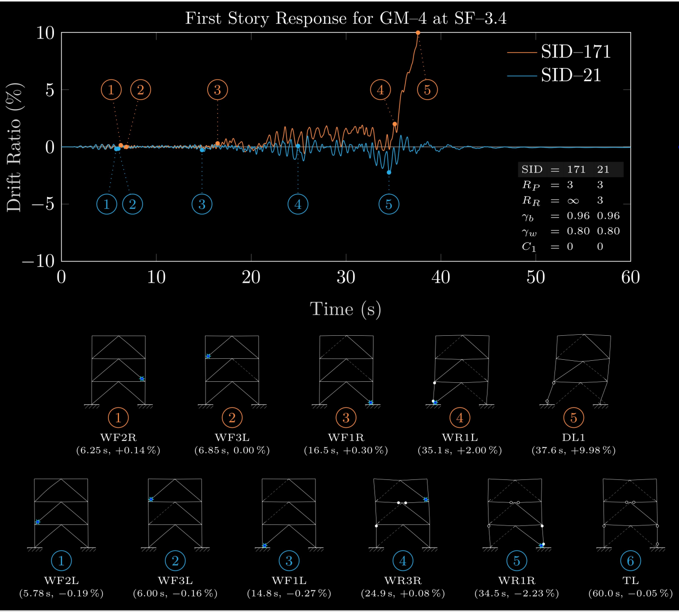
SID	R_P	R_R	γ_b	γ_w	C_1
173	5	∞	0.96	0.8	0
171	3	∞	0.96	0.8	0
21	3	3	0.96	0.8	0
3	3	3	0.64	0.8	0
57	3	3	0.64	1.8	0
138	3	3	0.64	1.8	1

Simulation Results



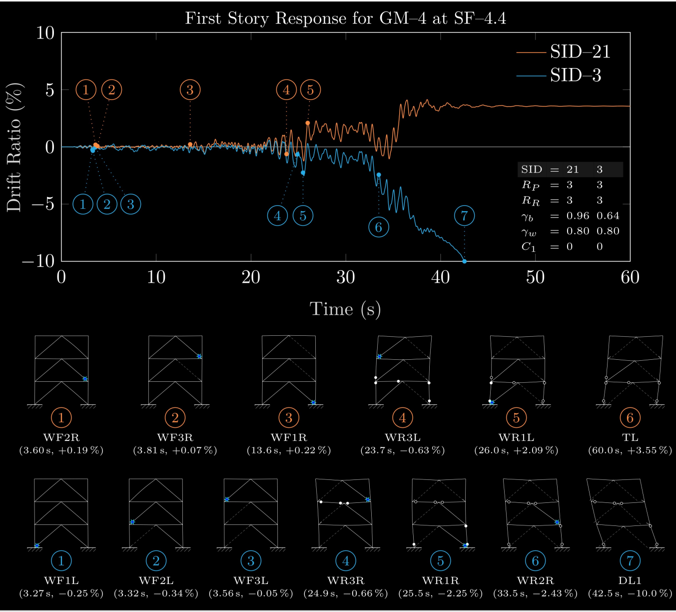
R_P

Simulation Results



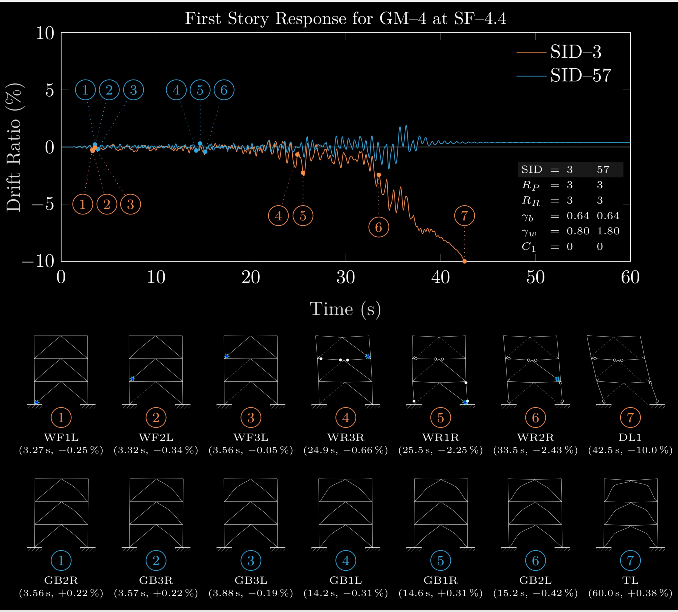
R_R

Simulation Results



γ_b

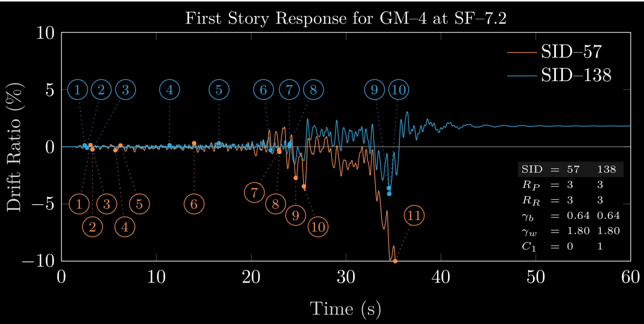
Simulation Results



SID	=	3	57
R_P	=	3	3
R_R	=	3	3
γ_b	=	0.64	0.64
γ_w	=	0.80	1.80
C_1	=	0	0

γ_w

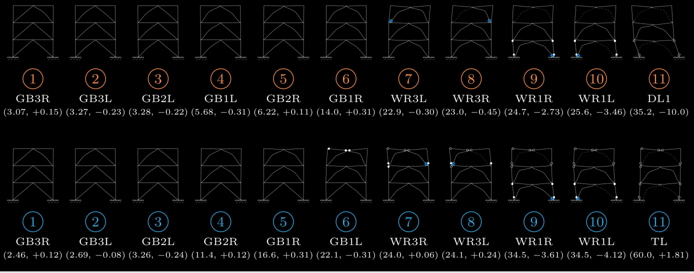
Simulation Results



First Story Response for GM–4 at SF–7.2

SID	=	57	138
R_P	=	3	3
R_R	=	3	3
γ_b	=	0.64	0.64
γ_w	=	1.80	1.80
C_1	=	0	1

C_1

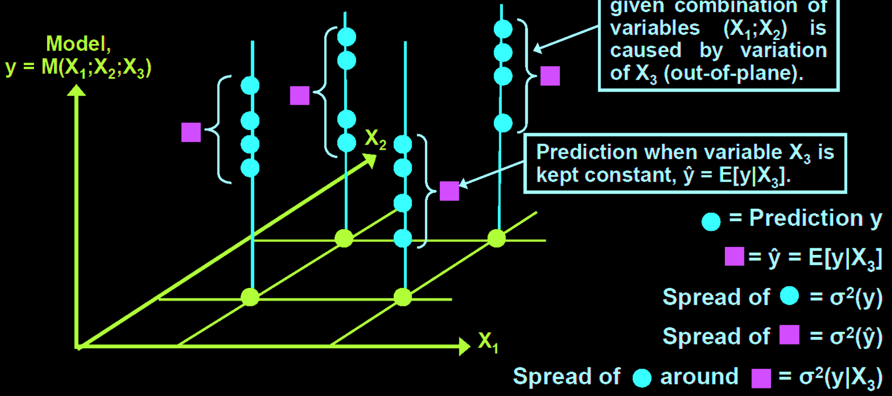


Simulation Results

Analysis of Variances (ANOVA)

- Statistical analysis method used to assess the relative influence of input parameters on the variation of a model output.
- In this work, the output of the ANOVA model was defined to be the probability of collapse determined from the fragility curves.
- This made it possible to quantify the relatively influence of the five design parameters on collapse performance.

Graphical Representation of ANOVA



This variability for a given combination of variables $(X_1;X_2)$ is caused by variation of X_3 (out-of-plane).

Prediction when variable X_3 is kept constant, $\hat{y} = E[y|X_3]$.

● = Prediction y

■ = $\hat{y} = E[y|X_3]$

Spread of ● = $\sigma^2(y)$

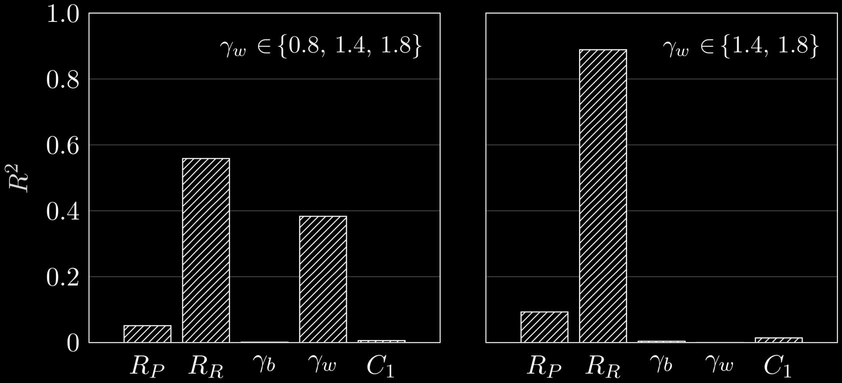
Spread of ■ = $\sigma^2(\hat{y})$

Spread of ● around ■ = $\sigma^2(y|X_3)$

Image reproduced with modification from Hemez (2006)

Simulation Results

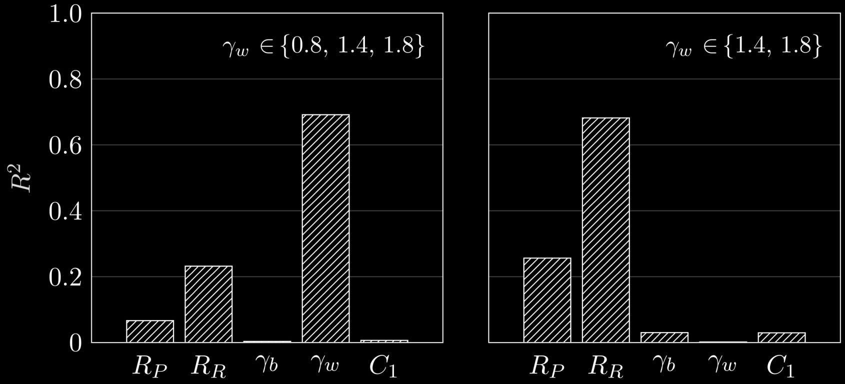
ANOVA



Correlation of P_F for ANOVA parameters of CBDF systems

Simulation Results

ANOVA



Correlation of P_F for ANOVA parameters of CBDF-R systems

Agenda

Introduction

Experimental Program

Parametric Study

Numerical Modeling

Simulation Results

Conclusions

Conclusions

How much ductility, how much strength, and how much reserve capacity are actually required for a building to survive an MCE event in a moderate seismic region?

Hines et al. (2009)

Conclusions

Ductility, Strength, and Reserve Capacity

- The only explicit ductility requirement in parameterized systems was the b/t limits.
- The b/t limits had relatively small impact on collapse performance, particularly for systems with deliberately-engineered reserve strength.
- Thus, it is argued that reserve strength ($R_R \leq 5$) can be more effective than ductility detailing to achieve controlled and favorable failure hierarchy within a moderate-seismic design philosophy.

Conclusions

In particular, can an inherently low-ductility system such as a concentric chevron braced steel frame be designed to survive moderate seismic demand with a high level of confidence?

Hines et al. (2009)

Conclusions

Confidence in Low-Ductility CBFs

- Due to cost-benefit, high-ductility detailing cannot be justified in moderate-seismic regions.
- Consequently, systems such as the $R = 3$ and OCBF are designed with little to no explicit ductility capacity and are prone to brittle limit states that are difficult to predict and control.
- Improved confidence can be achieved with economic viability if design requirements are defined to ensure favorable failure hierarchies while simultaneously providing reserve capacity.

Conclusions

Should such systems even be evaluated according to the concepts of ductility and capacity design?

Hines et al. (2009)

Conclusions

Appropriateness of Low-Ductility Design

- The $R = 3$ and OCBF are problematic because their performance capabilities are relatively unsubstantiated and uncertain.
- A more appropriate philosophy should be developed based upon the concepts of failure hierarchy and reserve capacity.
- Modifications to gravity framing components and connections that capitalize on existing strength provide a cost-effective method for achieving reserve strength in typical buildings.

Conclusions

> How can we articulate a design philosophy for low-ductility systems in moderate seismic regions that aims to ensure safety against collapse while allowing engineers as much freedom as possible to design creatively and economically?
>
> Hines et al. (2009)

Conclusions

Articulating a Moderate-Seismic CBF

- A balance between economic and performance considerations must be achieved.
- However, the low-ductility practice is misguided because it allows economic considerations to take precedence over protection of life-safety.
- A viable alternative must use simple design requirements to improve collapse performance without the need for extraneous detailing requirements or substantial changes to the traditional seismic design framework.

Conclusions

Final Recommendations

- Increase the brace KL/r limit to 200.
- Increase the brace b/t limit to $0.96\sqrt{E/F_y}$ for square HSS.
- Decrease the tension component of unbalanced loads to the unamplified load.
- Develop primary capacity on the order of a CBF with $R \leq 5$.
- Develop reserve capacity on the order of an MRF with $R \leq 5$.

Conclusions

Thank You!

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