Steel Structures Research Update

Inelastic Design Method for Steel Buildings Subjected to Wind Loads

JUDY LIU

INTRODUCTION

Ongoing research on an inelastic wind design method for steel buildings is highlighted. This study is under way at Brigham Young University, led by Dr. Johnn Judd, Assistant Professor in the Department of Civil and Construction Engineering. Dr. Judd's research interests include seismic performance of steel moment frame buildings, field evaluation and structural health monitoring of steel bridges, and wind performance assessment of buildings. Dr. Judd has also been awarded the AISC Milek Fellowship for this research on inelastic design methods for steel buildings subjected to wind loads. The research team is in the third year of the four-year study. Selected results from the study are highlighted along with a preview of future work.

RESEARCH OBJECTIVES AND PLAN

The major outcome of the research will be an inelastic design method for steel frame buildings subjected to wind loads. The methodology follows the ductile seismic design approach of controlled inelasticity in selected structural components and reduced design forces. Expected benefits include improved economy in design and system ductility for rare windstorms and high seismic events.

The basic research objectives are to establish loads, design prototype buildings, simulate nonlinear response, and evaluate reliability and risk. More specifically, the first objective is to identify steel buildings and develop the wind loads. This includes determining the duration of the wind loads and creating a set of archetype steel frame buildings. The second objective is to simulate nonlinear response with careful attention paid to structural damping and key modeling parameters. The third objective is to determine reliability and risk and develop guidelines for implementation of the inelastic design method.

Judy Liu, PhD, Research Editor of the AISC *Engineering Journal*, Professor, Oregon State University, School of Civil and Construction Engineering, Corvallis, Ore. Email: judy.liu@oregonstate.edu

The team is well into their research plan and is busy with design, simulation, and evaluation. In year 1, the research team determined the wind load characteristics needed for nonlinear response history analysis. The team also designed archetype steel frame buildings for further investigation in the study. Year 2 focused on identifying modeling parameters necessary for nonlinear response simulation of those archetype buildings subjected to wind loading. In year 3, the team has continued their design of prototype buildings while also conducting wind analyses. Wind load reliability and risk research will continue into year 4, followed by development of implementation guidelines.

IDENTIFY STEEL BUILDINGS AND WIND LOADS

First steps in the development of this inelastic method are to identify the steel frame buildings best suited for the design approach and to determine the wind loading for the study. A pilot study was first conducted to validate the proposed approach. This study also aided in the selection of the wind tunnel data and wind durations. An archetypal design space and prototype building designs were created.

Pilot Study

The pilot study was based on a structure with publicly available wind tunnel test data. The four-story, moment frame building is 78.5 ft long by 52.5 ft wide in plan and has a 13-ft story height, as shown in Figure 1. The lateral force-resisting system consists of a pair of 20-ft single-bay moment frames in each orthogonal direction. Ductile and conventional (i.e., nonductile) moment frame buildings were designed for a 110-mph wind. Exposure B was used to correlate to the suburban terrain used in the wind tunnel tests. Variable short duration and constant 4-hr wind events were considered.

Inelastic behavior was captured in the OpenSees (PEER, 2020) models of the prototype building. The ductile moment frame connections were based on the modified Ibarra-Medina-Krawinkler (IMK) model (Lignos and Krawinkler, 2011). The conventional moment frame buildings were assumed to have nonductile, flange-welded, pre-Northridge connections (ASCE, 2017; FEMA 2009, 2000) and were modeled using a Pinching4 model (Lowes et al., 2004). For column plastic hinges, the modified IMK model was adjusted to account for the influence of axial compressive load on the hysteretic behavior of the column (Lignos et al., 2019).

Evaluation of the prototype building under wind loading included nonlinear static wind pushover and incremental dynamic analysis. The incremental dynamic analysis was based on records from wind tunnel tests conducted at Tokyo Polytechnic University (TPU) (Tamura, 2012). The wind direction changed in 15° increments from a cross-wind direction (0°) to an along-wind direction (90°). The lateral load pattern for the pushover analysis mimicked the wind profile in open terrain (Exposure C). Figure 2(a) shows velocity profiles for suburban (Exposure B) and open terrain (Exposure C) along with the target profile for the wind tunnel tests. The wind speed at a height z above ground level, $V(z)$, is normalized by the basic wind speed, V. V is a 3-s gust for ASCE 7-16 (ASCE, 2017) and a 10-min average wind speed for the Japanese standard. The equivalent full-scale duration of the sampled wind tunnel tests was less than 1 hr; those records were replayed until the total wind event was equal to 4 hr (1-hr ramp-up, 2 hr at the target wind speed, and 1-hr ramp-down) as recommended in the *Prestandard for Performance-Based Wind Design* (ASCE, 2019). Figure 2(b) shows the time history of the pressure coefficient, C_p.

The results showed improved performance for the ductile moment frame compared to the conventional moment frame. In the pushover analyses for the ductile moment frame, there is a modest increase (14%) in system overstrength but a significant increase (250%) in system ductility, μ, compared to the conventional moment frame. From the incremental dynamic analysis, plots of wind speed versus maximum story drift ratio were generated. Figure 3 shows a median collapse wind speed of 172 mph for the conventional moment frame and 192 mph for the ductile moment frame. Collapse margin ratios (CMRs) used velocity pressure in comparisons of the median collapse intensity to design strength-level intensity; the CMRs are 2.4 and 3.0, respectively, for the conventional and ductile moment frames. This is a dramatic increase (by 25%) in the CMR for the ductile moment frame. Fragility curves were also used to assess probability of collapse. The analysis showed that the ductile moment frame designed for 110 mph provides the same level of capacity as a conventional moment frame designed for 123 mph. In terms of wind pressure, this represents a 25% increase in capacity.

The pilot study validated the proposed design approach and helped to define important parameters for the study. The preliminary results showed that a ductile moment frame could be designed with a wind force reduction factor, R_w, of 1.25 and meet the reliability of a conventional moment frame. The researchers also concluded that the ductile moment frame is capable of meeting the reliability target of 0.01% recommended in the *Prestandard for Performance-Based Wind Design* (ASCE, 2019). To capture effects of windstorm duration, 30-min. and 4-hr wind load records would be used. The team chose to base the

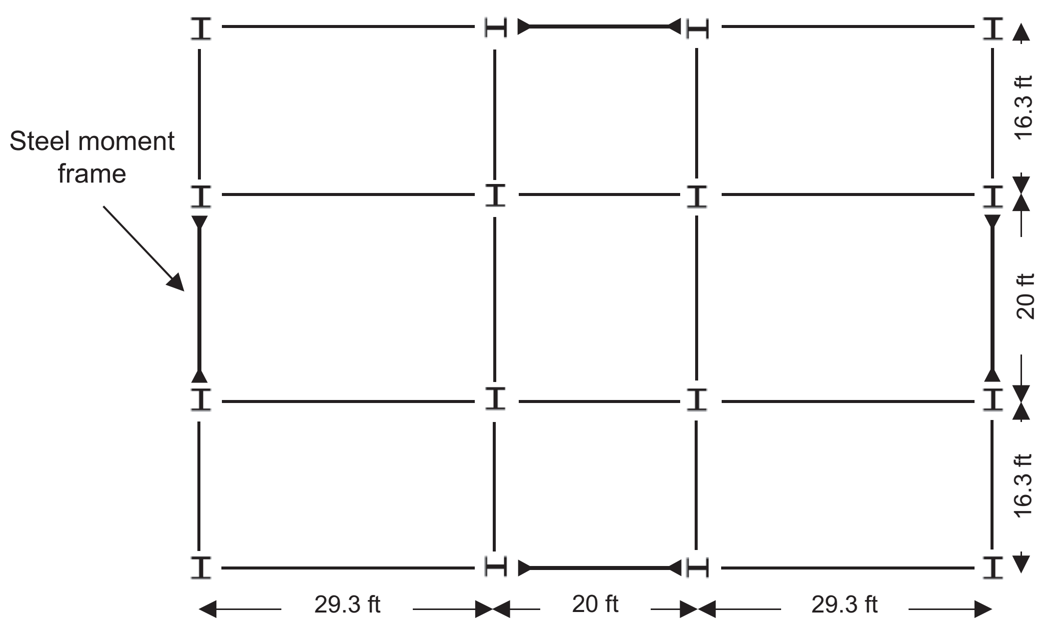


Fig. 1. Pilot study prototype building plan.

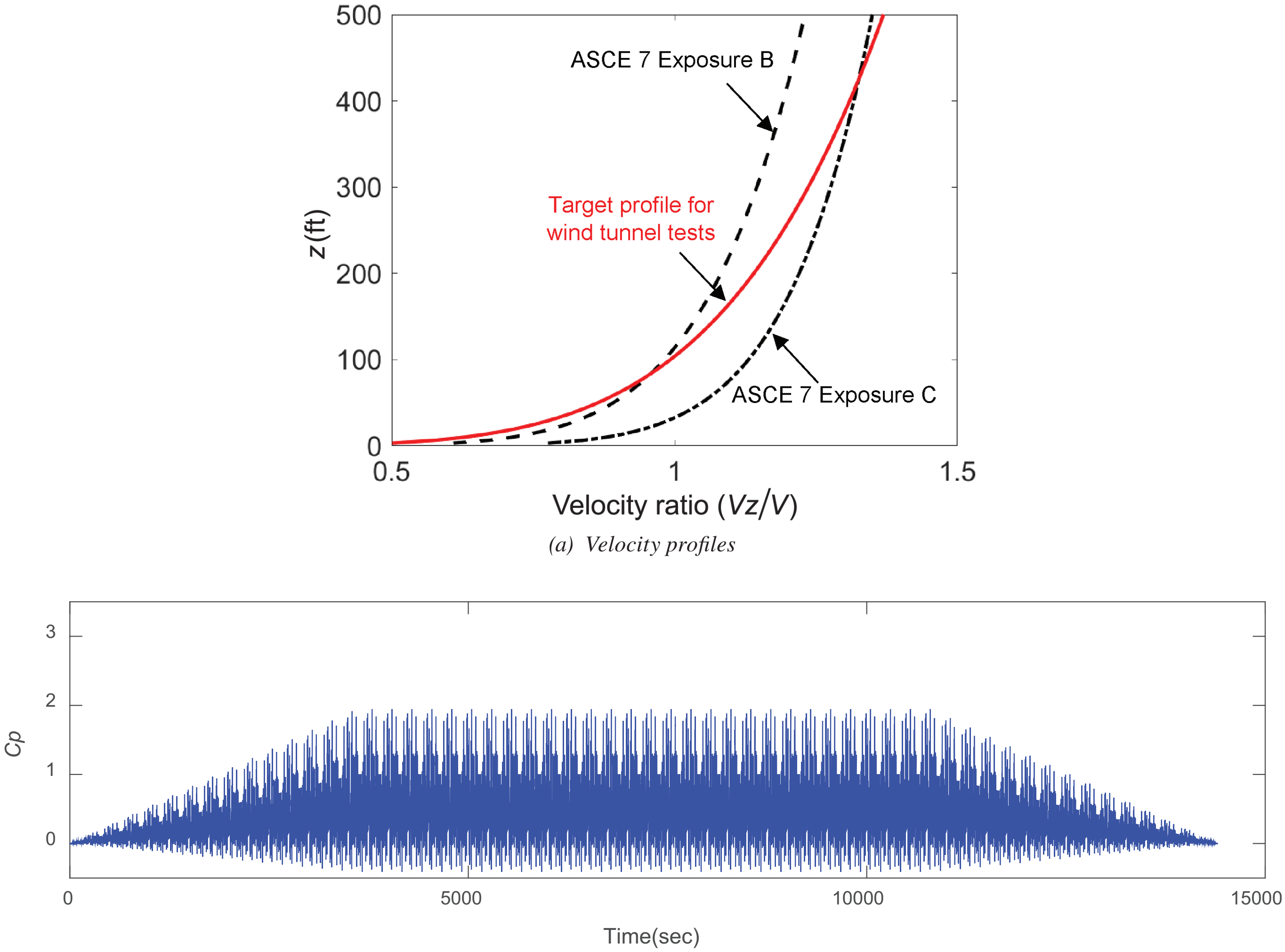
archetype buildings on a NIST database (NIST, 2017) instead of the TPU database to better correlate to the ASCE 7-16 exposure categories and utilize smaller increments in the wind direction angle.

Archetype Steel Frame Buildings

The design space for the archetype steel frame buildings ranges from type of lateral force-resisting system to the wind speed. Again, the buildings were partly based on structures with publicly available wind tunnel test data. There are also similarities to buildings used to illustrate the *Seismic Design Manual* (AISC, 2018). The building plans have typical aspect ratios and are suited to different lateral force-resisting systems.

There are three floor plans and two types of buildings with a few different heights [Figure 4(a)]. The one- and three-story office buildings are 125 × 80 ft in plan [Figure 4(b)]. The first story is 15 ft tall and the upper stories are 12 ft tall. There is also a 46-story office building at 100 × 150 × 600 ft. The one-story warehouse is 250 × 160 ft in plan with a 24-ft story height. Work to date has primarily been with the warehouse and the three-story office buildings.

Three types of lateral-force-resisting systems and levels of connection ductility are being evaluated. The lateral-force-resisting systems are steel moment frames (MF), steel-braced frames (BF), and composite plate shear walls filled with concrete (C-PSW/CF). Buildings have a pair of



(a) Velocity profiles

(b) Pressure coefficient time history

Fig. 2. Dynamic analysis of prototype building.

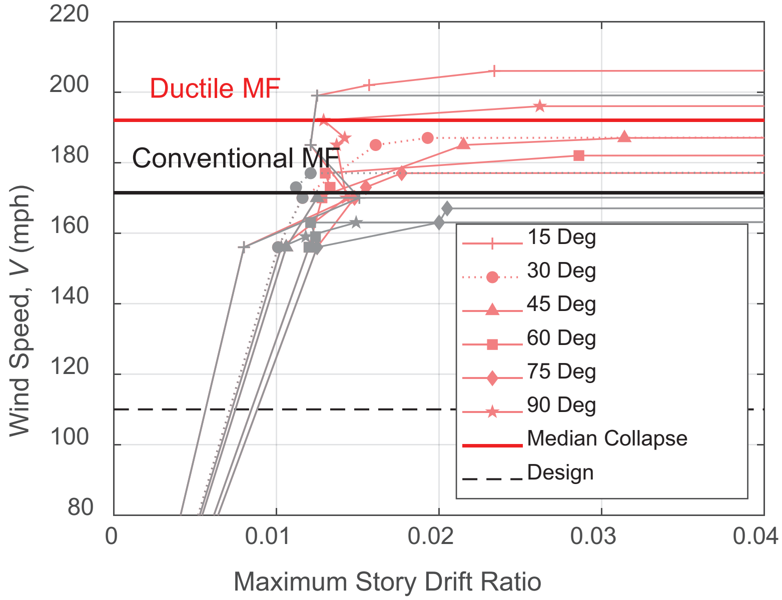
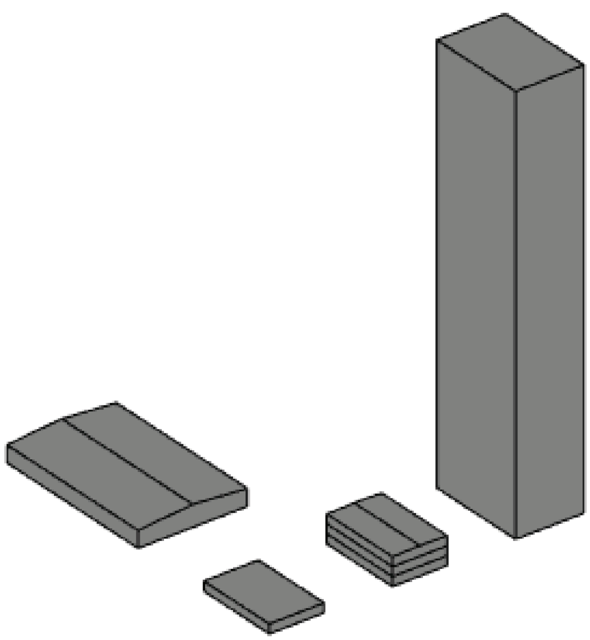
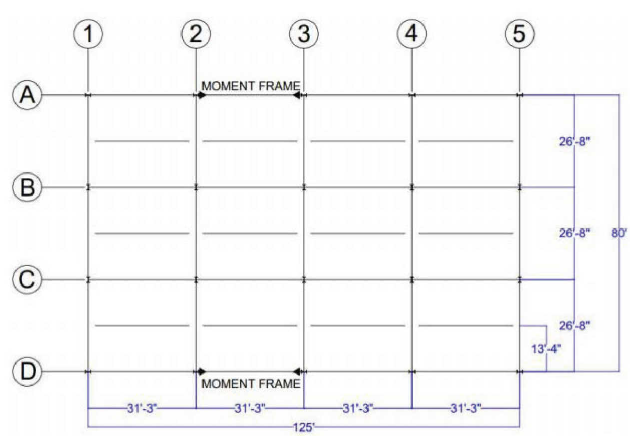


Fig. 3. Wind speed versus maximum story drift ratio for incremental dynamic analysis of prototype building.



(a) Schematics of archetype steel buildings



(b) One- and three-story office building floor plan showing moment frames only

Fig. 4. Steel-framed building archetypes.

moment frames in one direction and a pair of braced frames or composite shear walls in the orthogonal direction. Each lateral force-resisting system is designed using the directional procedure (ASCE, 2017). Design work on the braced frames and composite shear walls is in progress.

Low-, medium-, and high-connection ductility are considered for the moment frame designs. The low, or nonductile, moment frame connections are fully restrained (FR) flange-welded beam-to-column connections designed following the 2016 AISC *Specification* (AISC, 2016a). Moderately ductile frames use welded unreinforced flange-welded web (WUF-W) connections that satisfy the b/t limits for moderately ductile sections as defined in the 2016 AISC *Seismic Provisions* (AISC, 2016b). Highly ductile frames also used welded unreinforced flange-welded web (WUF-W) connections but satisfy the b/t limits for highly ductile sections (AISC, 2016b, 2016c).

The archetype buildings are designed for gravity, snow, and wind loads. The strength-level wind speeds are 110, 127, 156, and 220 mph in Exposure C. A height/400 drift limit is checked using a service-level wind speed. Floor and roof live loads are 80 and 20 psf, respectively. The snow load is 20 psf; 85 psf and 68 psf floor and roof dead loads are used for the one- and three-story office buildings. The curtain wall weight is taken as 175 lb/ft.

The steel moment frame designs were governed by strength criteria. Even at the lowest wind speeds, drift limits were satisfied with members sized for the flexural or combined axial and flexural loading. Preliminary design of the three-story office building resulted in member sizes ranging from W12×19 to W18×55 for the beams and W14×26 to W24×76 for the columns. Preliminary member sizing was followed by panel zone shear and flange local bending checks. Additional details of the moment frame designs can be found in Gocke (2020) and Giles (2021) for the warehouse and office buildings, respectively.

The braced frame designs have also been governed by strength criteria. For the warehouse building, the square HSS members used for the diagonal braces were typically near 90% of their strength capacity. Member sizes ranged from HSS4×4×0.25 to HSS8×8×0.313. At higher wind speeds, local demands on the beams from the chevron braces required upsizing of the W-shapes for the limit state of flange local bending or local web yielding. Additional details of the braced frame designs can be found in Jacobs (2020).

SIMULATION OF NONLINEAR BEHAVIOR

Next steps were focused on simulation of the nonlinear behavior of the archetype buildings. In the proposed design approach, the premise is that controlled inelasticity and structural ductility allows for reduced design forces and improves system ductility. The control of inelasticity and ductility is at multiple levels: material response, local buckling, and selected components. Material ductility can be preserved by limiting the yield-to-ultimate tensile stress. Local buckling can be delayed by limiting the width-to-thickness ratio of section elements. Locations of inelasticity are limited to selected components. As such, the various sources of inelasticity are important for modeling and evaluation of the archetype buildings. The second year of the project addressed key modeling parameters for the steel systems. The steel moment frame and steel concentrically braced frame modeling are briefly highlighted here.

Steel Moment Frame Models

The sources of inelasticity in the steel moment frame models include ductile and nonductile beam plastic hinges. Figure 5(a) shows the ductile beam plastic hinge formation, again idealized using the modified IMK hysteresis model for the moment-rotation response. The cyclic degradation due to local buckling is considered directly in the rule-based hysteresis model. Parameters used in the model are based on statistical analysis of test data (Lignos and Krawinkler, 2011). Figure 5(b) shows the nonductile beam plastic hinge. For this model, a trilinear loading and unloading rule-based hysteresis model for reinforced concrete moment frames (Lowes et al., 2004) was adapted to capture fracture and pinching in the cyclic response. Parameters used for the nonductile beam plastic hinge were based on the behavior of pre-Northridge, flange-welded connections as evaluated by Lee and Foutch (2002) and others.

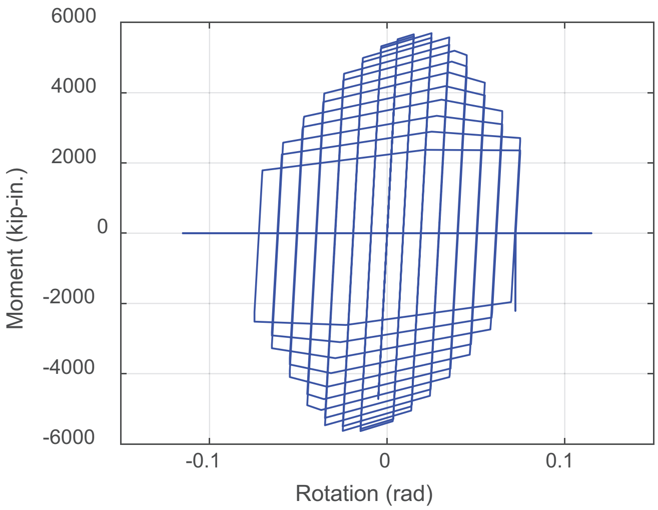
The column plastic hinges and panel zones are additional sources of inelasticity. The column plastic hinges are also idealized using the IMK hysteresis model. The modeling parameters account for the influence of axial compressive loading following Lignos et al. (2019). The cyclic shear force-deformations in the column panel zones are modeled using the Krawinkler model (Krawinkler, 1978). Additional details of the modeling can be found in Giles (2021).

Steel Concentrically Braced Frame Models

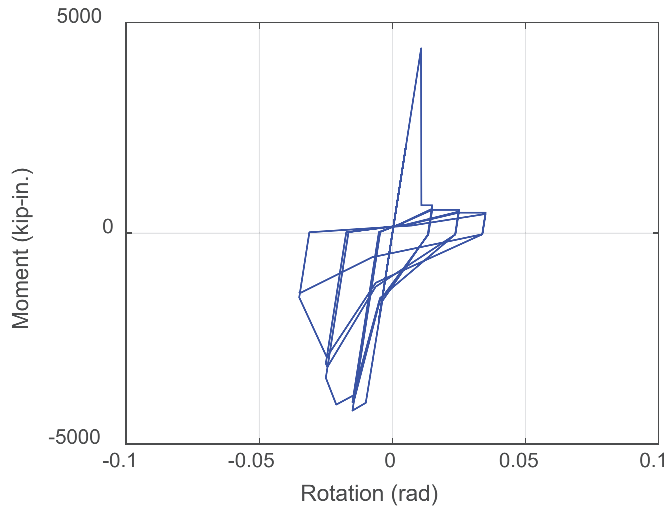
The steel concentrically braced frame models use distributed and concentrated plasticity. The beams, columns, and braces were modeled with nonlinear beam-column elements with fiber sections, as shown in Figure 6. The diagonal braces are subdivided into 10 elements with an out-of-plane initial imperfection or out-of-straightness. The imperfection was set to length/500 to achieve the predicted axial compressive strength calculated following the AISC *Specification* (AISC, 2016a). The fiber sections are used to capture local behaviors. The brace cross sections, for example, have four fibers across a side and four fibers through the thickness. The material modeling includes isotropic strain hardening and fatigue based on work by Uriz (2005) for steel braced frames. Shear tab and gusset plate connections are modeled using nonlinear spring elements and parameters as defined in the literature (e.g., Hsiao et al., 2012).

Finite Element Analysis

The building responses are evaluated with cyclic nonlinear static pushover and nonlinear dynamic response history analysis. Cyclic nonlinear static pushover analysis is used to obtain system parameters such as overstrength and ductility. The lateral load pattern corresponds to the Exposure C wind profile and is applied with a displacement-control strategy at a roof node. Figure 7 shows a representative cyclic pushover curve. System overstrength for wind, Ω_{wind}, is defined as the maximum base shear force in the pushover analysis, F_{max}, divided by the wind design base shear force, F_{design}. The ductility, μ, is obtained by dividing the post-peak roof displacement at 80% of the peak load, $\Delta_{80}\%$,



(a) *Ductile beam plastic hinges*



(b) *Nonductile beam plastic hinges*

Fig. 5. Steel moment frame models.

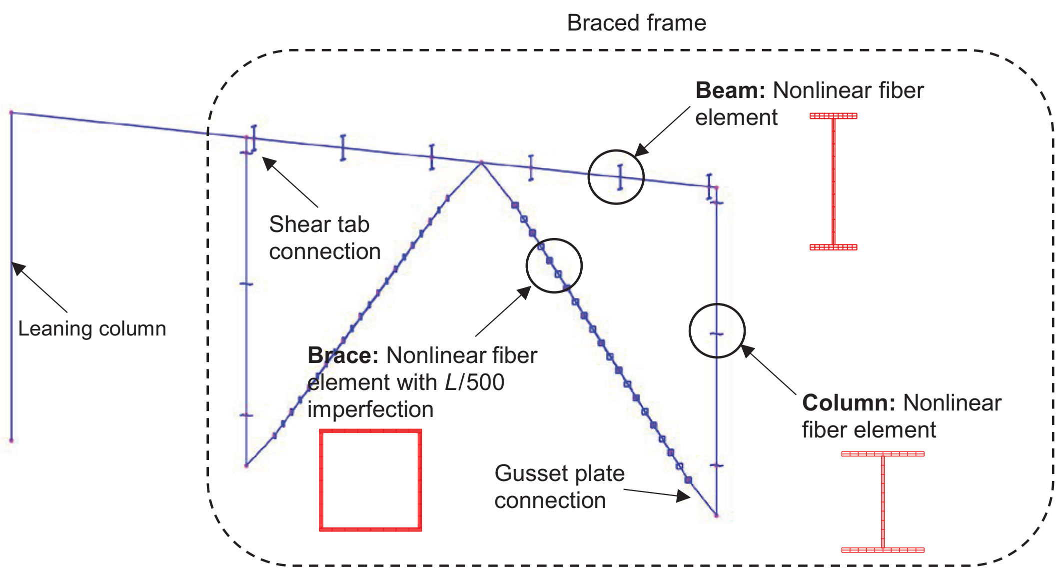


Fig. 6. Braced frame model with beam, brace, and column fiber elements.

by the effective yield displacement, Δ_y, (FEMA, 2009). Details on the cyclic nonlinear static pushover and nonlinear dynamic response history analysis can be found in Giles (2021).

Preliminary Results

Results from the cyclic nonlinear static pushover analyses have further validated the proposed approach. The effects of controlling inelasticity by limiting width-to-thickness ratios and by limiting inelasticity to selected components are investigated. For the braced frame in the warehouse building, the cyclic response for a conventional beam and non-slender brace, shown in Figure 8(a), is compared to a design with a strong beam and highly ductile brace, as shown in Figure 8(b). The system overstrength for wind, Ω_{wind}, increases from 2.5 to 10.6 for the strong beam and highly ductile brace. The ductility, μ, decreases from 40 to 23. These and other results demonstrate the effectiveness of connection ductility and controlling inelasticity

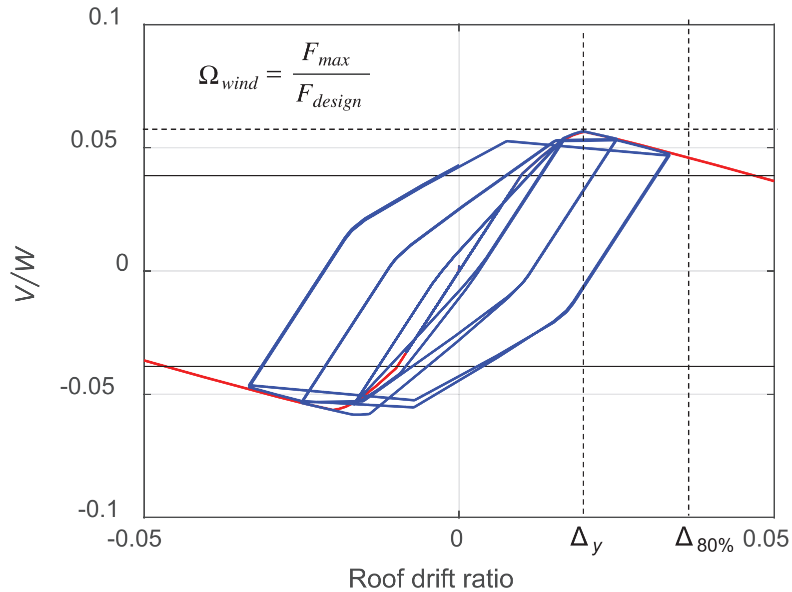
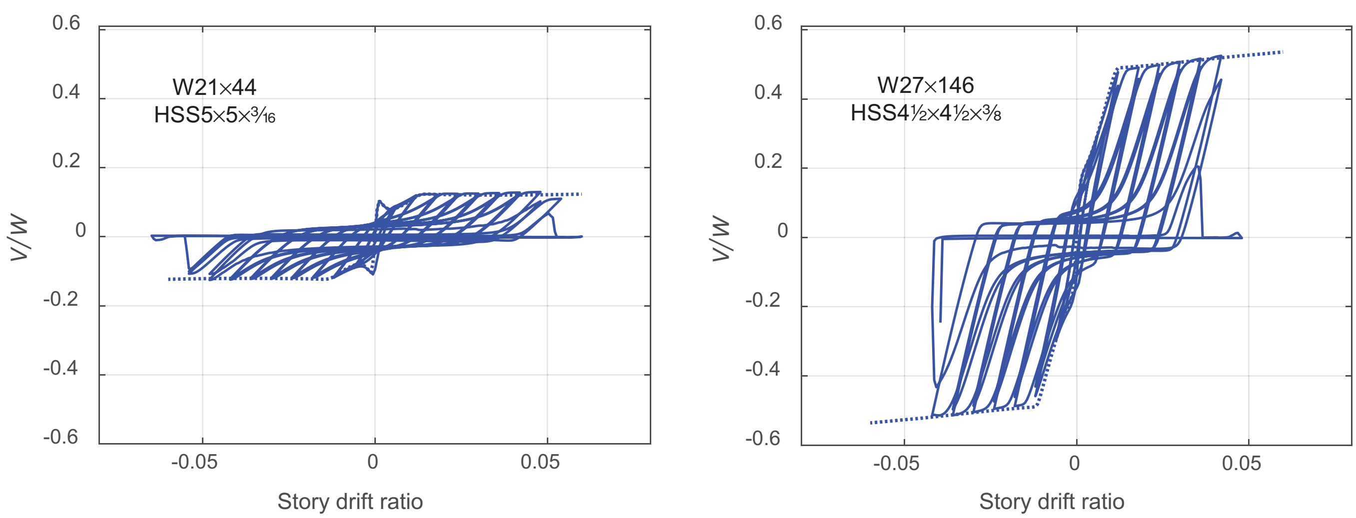


Fig. 7. Representative cyclic nonlinear static pushover analysis.



(a) Conventional beam and nonslender brace

(b) Strong beam and highly ductile brace

Fig. 8. Comparison of cyclic pushover response of braced frame in the warehouse building for 110-mph design.

for improving the predicted global strength of the system subjected to wind loads. The research team expects similar observations for the incremental dynamic analysis.

RELIABILITY AND RISK, FUTURE WORK, AND EXPECTED OUTCOMES

The third research objective centers around reliability and risk and forms the bulk of the future work. The main tasks are to provide a comprehensive analysis of reliability and risk for inelastic wind design and to develop guidelines for practical implementation of the proposed design approach. Designs of the concentrically braced frame (CBF) and composite plate shear wall filled with concrete (C-PSW/CF) systems will be finalized for the office buildings. The designs will be evaluated through cyclic nonlinear static pushover analysis, nonlinear dynamic response history analysis, and incremental dynamic analysis (Figure 9). Expected outcomes include recommendations for wind strength modification factors and best practices for controlling inelasticity.

ACKNOWLEDGMENTS

Thank you to Dr. Johnn Judd for his many contributions to this article. The work of graduate students from the University of Wyoming (Josef K. Jacobs, Jonah W. Gocke, S. M. Ashfaqul Hoq) and Brigham Young University (Tyler E. Giles, Conrad Belshe) is also recognized. The research is sponsored by the American Institute of Steel Construction (AISC); Devin Huber, Director of Research, is the project manager. The researchers would like to thank AISC Research Oversight Committee members Patrick McManus, Bonnie Manley, Lee Shoemaker, and Joseph Zona. Any findings or recommendations are those of the researchers and do not necessarily reflect the views of the sponsor.

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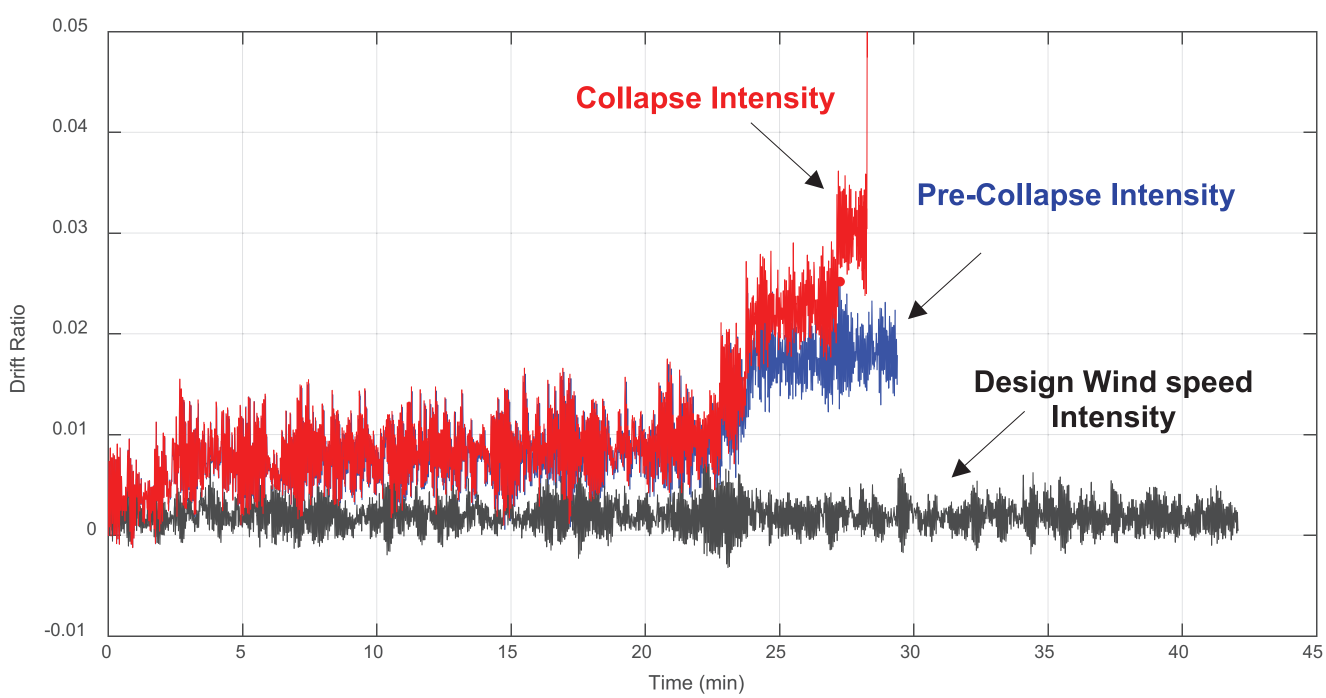

Fig. 9. Design, pre-collapse, and collapse intensity results from incremental dynamic analysis.

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