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IINELASTIC WIND DESIGN OF A STEEL CONCENTRICALLY BRACED FRAME BUILDING

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Abstract. An inelastic wind design method for steel frame buildings that allows limited inelasticity under strength-level loads is presented in this paper. In the proposed design method, the main wind force resisting system is designed using reduced strength-level design forces by employing a wind response modification factor that is based on ductility and overstrength. Serviceability requirements are still met using elastic design. The proposed design method is illustrated for a 3-story building with steel concentrically braced frames. Two types of braced frames were designed: a conventional braced frame based on an elastic design and a ductile braced frame based on the proposed inelastic design method. Incremental dynamic analysis results show that the collapse capacity of the inelastically designed braced frames exceed the collapse capacity of the conventional braced frame. Both the conventionally designed and inelastically designed braced frames have a low estimated risk of wind collapse.

1 INTRODUCTION

1.1 Background

Safe and economical design drives innovation in structural engineering. While keeping safety paramount, efforts to reduce over-conservative designs lower the financial and environmental cost of projects, thereby increasing efficiency. Specifically, when designing a structure, member sizes are determined in response to applied design loads. If design loads can be reduced, the structural member design will typically be reduced as well.

The conventional approach in the structural design of lateral systems in buildings is to allow inelasticity for seismic load applications. Inelastic seismic design is based on the equal displacement concept. The equal displacement concept expresses the idea that both elastically and inelastically designed lateral systems experience similar amounts of displacement when subjected to ground motions [1]. This phenomenon applies to ground excitation of structures with long natural periods (velocity and displacement sensitive structures) where second-order effects do not cause system instability [2]. Due to the equal displacement concept, loads induced on a structure by seismic activity may be reduced using a seismic response modification coefficient, R, which is based on the response of a given lateral system. The use of R for seismic design has been well studied and understood in industry for a variety of lateral systems (e.g. [3,4]). However, no such response modification factor is currently used for wind design of lateral systems. One reasons for this is that wind loads do not have a mean load of zero. As a consequence, while seismic oscillations cause roughly equal positive and negative displacements in a given direction, along wind loads produce positive displacements. Thus, seismic displacements are essentially symmetric while wind displacements are not. The upshot is that the equal displacement concept does not hold under wind loads [2]. However, since the response modification factor also incorporates overstrength in the lateral system, it is possible that a wind response modification factor, R_{wind} could be used in wind design. This is important because an inelastic wind design method has the potential to be more economical for rare wind events and more desirable in regions where the building is exposed to a combination of high-seismic and high-wind hazards by ensuring that selected ductile components (structural "fuses") protect the surrounding structure.

Braced frames are an important type of lateral system in buildings. A braced frame mainly resists lateral seismic or wind loads through diagonal bracing members (braces). The braces in a braced frame resist these lateral loads through axial tension and or axial compression. One common type of a steel braced frame is a concentrically braced frame. In a concentrically braced frame, the beams, columns and braces form a vertical truss that resists lateral forces by truss action. A concentrically braced frame can develop ductility through inelastic action in braces when the braces yield in tension and when the braces buckle in compression. Compared to other lateral systems (e.g. steel moment frames and buckling restrained braced frames), a concentrically braced frame tends to be less ductile but it has higher elastic stiffness, which is an important characteristic for serviceability (e.g. story drift control) under wind loads.

The hypothesis of this study is that main wind force resisting system can be designed with reduced design forces using a wind response modification factor based on ductility and overstrength, similar to the approach that is used for seismic design, by preserving material ductility, delaying local buckling, limiting inelasticity to selected components, and generating system-level overstrength through material overstrength, design overstrength, and redistribution of lateral load.

1.2 Objective

In this study, an inelastic design method for wind was developed for a ductile braced frame. The proposed inelastic design method is presented first. The proposed method is then applied to a 3-story building with steel concentrically braced frames. For this building, two types of braced frames were designed: a "conventional" braced frame based on an elastic design, and a ductile braced frame based on the proposed inelastic design method. Finite element models were used to simulate the inelastic response of the main wind force resisting system. Nonlinear static "pushover" analysis and incremental dynamic response history analysis of the finite element models were used to determine the static wind collapse capacity and wind collapse fragility. The wind collapse fragility was integrated with location-based wind hazard data to estimate system-level safety and reliability.

2 INELASTIC DESIGN METHOD

2.1 Design Wind Loads

Wind loads are caused by the local flow of air around a building. Wind loads are affected by separations, reattachments, and vortices in the wind flow, which are directly related to the shape of the structure. Compared to the effect of building shape and size, the effect of upwind terrain on the wind load is relatively small. The pressure generated by wind can be determined using Bernoulli's equation for fluid flow:

$$q = \frac{1}{2}\rho V^2 \tag{1}$$

where q is the velocity pressure, ρ is the mass density of air, and V is the wind velocity. The velocity pressure equation is then adjusted for height, terrain condition, and the aerodynamic characteristics of the building. Currently in the United States, design wind loads for the lateral system (the main wind force resisting system) and the aerodynamic characteristics of the building are determined using one of three procedures: the directional procedure, the envelope procedure, and the wind tunnel procedure.

- In the directional procedure, design loads are based on external pressure coefficients that are applied normal to the wall or roof surface for specific wind directions.

- In the envelope procedure, design loads are based on pseudo-external pressure coefficients that are applied normal to the wall or roof surface. The pseudo-external pressure coefficients were derived by rotating small-scale models of buildings 360 degrees in a wind tunnel while simultaneously monitoring the wall and roof pressures. The magnitude of the external pressure coefficients that envelope the maximum structural response (uplift forces, base shear, and bending moments in building frames) were recorded and used to determine the pseudo-external pressure coefficients.

- In the wind tunnel procedure, the design loads are based on the pressures recorded on a scale-model of the specific building that is being designed.

The wind tunnel procedure is primarily used for flexible structures, irregularly shaped buildings or buildings shielded by adjacent buildings, whereas the directional and envelope procedures are used for routine design. Therefore, this study used the directional procedure.

2.2 Elastic Design

The conventional design of braced frame relies on elastic design [5] without special ductile detailing or member proportioning requirements. The brace may be designed to resist both axial tension and axial compression, or it may be designed to resist only tension. If it is designed to resist both forces, compression will control. The limit states that control the strength of the brace include compression or tension in the brace, and the strength of bolted or welded gusset plates, among other limit states. The size required to support gravity loads can be used to determine the initial sizes for the beam and column in a braced frame. These sizes are then typically increased based on the lateral loads.

Although conventional braced frames are sometimes termed "non-ductile" they exhibit ductility. For this reason, the seismic response modification factor is 3 (a factor equal to 1 would imply that the system has no ductility).

2.2 Inelastic Design

In the proposed design method, the main wind force resisting system is designed using reduced strength-level design forces by employing a wind response modification factor. The inelastic behavior is restricted to the braces. The braces are to be the structural fuses in a concentrically braced frame, while beams and columns are intended to remain more or less elastic. This approach is an adaptation of the approach that is currently used in the United States for seismic design [6]. To achieve this type of behavior,

1) a steel with a low yield-to-ultimate stress (F_y/F_u) ratio should be used,

2) brace members with good energy dissipation capacity and fracture life should be used to delay local buckling and distortional buckling prior to achieving the desired inelasticity, e.g. members capable of 0.02 rad plastic rotation, and

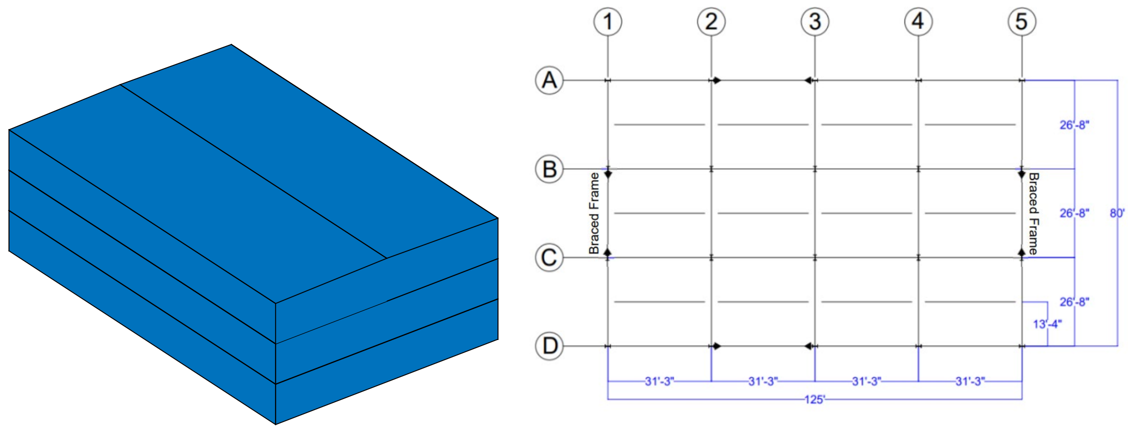
3) brace connections, beams and columns (and column splices and column bases) should be designed for the maximum forces and deformations imposed by the brace. In steel braced frames, the expected brace strength is $R_y F_y A_g$, where A_g is the gross cross section, F_y is the specified minimum yield stress, and R_y is the ratio of expected to specified yield stress.

4) in a braced frame with an inverted-V chevron configuration, the beam should be capable of resisting the expected brace strength in compression $(1/0.877)^* F_n A_g$, where F_n is the critical inelastic/elastic flexural buckling stress defined in AISC 360 Chapter E [5]. Importantly, the beam need not be capable of resisting the expected brace strength in tension ($R_y F_y A_g$).

3 APPLICATION

3.1 Braced Frame Design

The proposed design method is illustrated for a 3-story building with steel concentrically braced frames in Summerville, South Carolina. Figure 1 shows a perspective view of the building and the framing layout. The building is 80 feet wide by 125 feet long by 40 feet tall. The building matches the "NIST im1" building that is described in the National Institute of Standards and Technology aerodynamic database (https://www.nist.gov/el/materials-and-structural-systems-division-73100/nist-aerodynamic-database). The braced frames are located in the middle bays of the longer dimension (the transverse direction). The braced frame is in X-bracing configuration for the first two stories, and in an inverted-V chevron for the third story.



(a) (b)

Figure 1: The 3-story building with steel concentric braced frames: (a) perspective view, and (b) framing layout.

The building was designed for a 156-mph basic strength-level wind speed (Risk Category IV wind speed). Two types of braced frames were designed: a conventional braced frame based on an elastic design and braced frames based on the proposed inelastic design method. For the inelastic design method, two values of R_{wind} were examined: 1.5 and 2.0. Table 1 gives the resulting member sizes and corresponding demand-to-capacity ratio (DCR) for each member.

3.2 Finite Element Model

The braced frames were modeled in OpenSees finite element analysis software [7]. A hybrid distributed-plasticity and concentrated-plasticity approach was used. The beams, columns, and braces in the frame were modeled using nonlinear beam-column elements with fiber sections (distributed plasticity), while beam-to-column connections, shear tab connections, and gusset plate connections were modeled using nonlinear springs (concentrated plasticity).

Table 1:Braced frame member sizes and demand-to-capacity ratio (DCR).

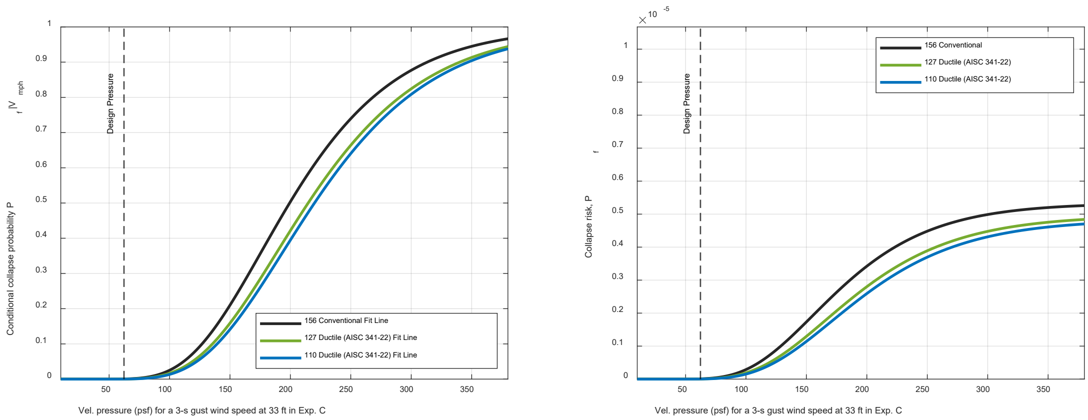
Story	Member Size			DCR		
	Brace	**Beam**	**Column**	**Brace**	**Beam**	**Column**
Elastic (Conventional) Design						
3	HSS 4 x 4 x 1/8	W12x26	W14x48	0.93	0.99	-
2	HSS 4 x 4 x 1/2	W24x76	W14x48	0.99	0.92	-
1	HSS 6 x 6 x 3/16	W12x35	W14x48	0.92	0.94	0.92
Inelastic Design with R_{wind} = 1.5						
3	HSS 3-1/2 x 3-1/2 x 1/4	W27x84	W14x48	0.54	0.86	-
2	HSS 4 x 4 x 5/16	W24x76	W14x48	0.86	0.92	-
1	HSS 4-1/2 x 4-1/2 x 5/16	W21x57	W14x48	0.94	0.98	0.85
Inelastic Design R_{wind} = 2.0						
3	HSS 3 x 3 x 3/16	W12x65	W14x48	0.83	0.94	-
2	HSS 3-1/2 x 3-1/2 x 3/8	W24x76	W14x48	0.91	0.97	-
1	HSS 4 x 4 x 3/8	W21x68	W14x48	0.92	0.97	0.73

Nonlinear static "pushover" analyses of the braced frame models were used to determine the static wind collapse capacity, system overstrength, system ductility, and the side-sway failure (story) mechanism. In the pushover analysis, the frame was subjected to a lateral load pattern that was proportioned to match the lateral loads in a suburban terrain (Exposure C). The lateral loads were applied using a displacement-control solution strategy.

Incremental nonlinear dynamic response history analyses were used to determine the wind collapse fragility. In the dynamic analysis, the frame was subjected to lateral wind loads that were derived from the wind tunnel test data in the National Institute of Standards and Technology aerodynamic database. A total of 37 wind angles were run, ranging in increments of five degrees. The original wind tunnel records are short-duration wind records. These records were modified to simulate an approximately 4-hour windstorm by replaying and scaling the wind tunnel record. The modified record contained, approximately, a 1-hour ramp-up segment, a 2-hour constant segment, and a 1-hour ramp down segment. For each wind angle, the analysis was run at a given intensity and the response (i.e. story drift) was recorded. The analysis was then run again at an incrementally higher intensity. This was repeated until the building collapsed. The collapse points (collapse intensities) were fit with a lognormal cumulative distribution function to create a wind collapse fragility curve. The slope of the lognormal cumulative distribution function was decreased to account for aleatoric and epistemic uncertainties. The wind collapse fragility was integrated with location-based wind hazard data to estimate system-level safety and reliability. This process is described in detail elsewhere [8].

3.3 Results

The results are shown below. Figure 2(a) shows the conditional wind collapse fragility. The median collapse pressure for all the frames is approximately four times the strength-level design pressure. The collapse capacity of the inelastically designed braced frames exceed the collapse capacity of the conventional braced frame. Figure 2(b) shows the unconditional collapse risk for the building location (Summerville, South Carolina). In terms of reliability, both the conventionally designed and inelastically designed braced frames have a low estimated risk of wind collapse. The risk of collapse meets the target in the ASCE 7 *Prestandard for performance-based wind design* [9] of $5.0\text{x}10^{-6}$ and corresponds to a reliability index, β of 4.4.



(a) (b)

Figure 2: Analysis results for the 3-story building: (a) wind collapse fragility, and (b) wind collapse risk.

4 CONCLUSIONS

An inelastic wind design method for steel frame buildings was developed that allows limited inelasticity under strength-level loads based on the ductility and overstrength of the main wind force resisting system. The proposed design method was applied to the design of a 3-story building with steel concentrically braced frames and compared to a conventional elastic design. Nonlinear response history analysis results show that, for the building examined in this study with steel concentrically braced frames, the wind collapse capacity and wind collapse risk of the inelastically designed braced frames met or exceeded the conventionally designed braced frame.

These findings suggest that some inelasticity could be safely permitted for wind design of the main wind force resisting system. However, a wind response modification factor greater than 1.0 should not be used when designing building components and cladding, or for serviceability limit states. Although this paper focuses on steel braced frames, the proposed method is intended to be broadly applicable (e.g. to moment frames, shear walls) when the lateral system is stiff (strength-controlled).

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