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Storm duration effects on roof-to-wall-connection failures of a residential, wood-frame, gable roof

T.K. Guha, Gregory A. Kopp *

Boundary Layer Wind Tunnel Laboratory, Faculty of Engineering, University of Western Ontario, London, Ontario, Canada N6A 5B9

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A B S T R A C T

This paper investigates the storm duration effects on roof-to-wall-connection failures of a wood-framed, residential structure using a combined load sharing, nail-slip model. A model of the roof structure, idealized as an equivalent 2-D beam for the vertical load path under wind-induced uplift, is used in combination with piecewise-linear load–displacement curves for the roof-to-wall-connections. The model was validated against experimental data for a system of interconnected roof-to-wall-connections subjected to fluctuating wind loads. The validated model, subjected to fluctuating wind loads obtained from design storms of different durations along with the stochastically sampled connection parameters established from experiments, was used to estimate the probability of roof failures of a gable roof building. It was found that for "design-level" storms with high winds over durations of 1–5 h, the probability of failure was increased by 15% for the longer storm, even though the peak pressures were held constant for these storms.

1. Introduction

Post-event damage surveys have found that the most vulnerable structural components in wood-frame houses are the roofs, caused by large suctions and weak links in the vertical load paths. Roof sheathing and toe-nailed, roof-to-wall-connections (RTWCs) are particularly vulnerable (e.g., Van de Lindt et al., 2007; Prevatt et al., 2012; Amini and van de Lindt, 2013; Morrison et al., 2014). Fig. 1 depicts one such example of failure of the RTWCs.

Analyses of both roof sheathing and global roof failures have been carried out in the context of North American housing by many authors using experimental and analytical/numerical studies. Reed et al. (1997), Cheng (2004), Ellingwood et al. (2004), and Shanmugam et al. (2009), among others for example, have examined the capacity of both individual and in-situ toe-nail connections to uplift wind loads, as well as various retrofit strategies such as the use of commercial metal straps to strengthen the connections. He et al. (2001) and Dao and van de Lindt (2008) used non-linear finite element modeling coupled with a mechanics based load–deformation approach to study the response of wood-fasteners and timber-framed buildings subjected to wind loads. He (2010) applied finite element modeling to predict the uplift capacity of the roof sheathing subjected to different fastener spacing under fluctuating wind loads. Ellingwood et al. (2004) and Lee and Rosowsky (2005) carried out fragility assessments for roof sheathing failures in high wind regions. In particular, a sensitivity analysis of the building type (eaves height and roof features such as overhangs), exposure, spacing of fasteners, etc., on the wind fragility for both roof sheathing and roof-to-wall connections were presented. More recently, Amini and van de Lindt (2013) analyzed failures of roof sheathing and roof-to-wall-connections in the context of tornadoes. All of these analyses consider failures to be due to a static, single peak in the wind load, with the notable exception of He (2010).

Of course, real storms generate a series of peak loads of varying magnitudes. These peaks are due to fluctuating (turbulent) wind in the atmospheric surface layer, together with the building aerodynamics, which yield spatially and temporally varying pressures on all building surfaces. Both the magnitude and distribution of wind loads on a roof depend on many parameters, such as the building shape, roof shape, the surrounding terrain and immediately surrounding buildings, as well as wind velocity and wind direction. Building codes and typical structural analyses handle the spatial variations by separating the building surfaces into a few zones with equivalent uniform loads in each zone. Temporal variations are largely neglected and peak, short duration gust loads are used for design of non-resonant structures (such as wood-frame houses) and for most components. Thus, storm duration effects are essentially neglected in both design and the assessment of damage and risk for wood-frame houses.

Recent research has found that both roof sheathing (Henderson et al., 2013b) and toe-nailed RTWCs (Morrison and Kopp, 2011; Morrison et al., 2012; Henderson et al., 2013a) tend to fail by

* Corresponding author.
E-mail address: gakopp@uwo.ca (G.A. Kopp).

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incremental damage accumulation caused by the severe short duration peaks of the applied wind load. Post-storm damage investigations (Ginger et al., 2007; see also Kopp et al., 2012) have also identified such progressive damage of nailed roof connections. One implication of this is that the overall duration of the storm plays a role in such failures, with longer storms providing larger numbers of the large magnitude, short duration peaks in the pressures necessary to cause ultimate failure.

In the general context of wind loading studies of structures, responses are usually considered to be in the elastic range. Few studies have addressed the issue of yielding behavior and damage accumulation of structures to fluctuating wind loads; notable exceptions being those by Vickery (1970) and Georgiou et al. (1988). Both of these studies have considered the narrow band, dynamic, resonant behavior of non-linear, inelastic single-degree-of-freedom (SDOF) steel structural components/elements to time varying wind loads. It is also to be noted that studies of low cycle fatigue (Henderson et al., 2009), and load cycles on glass (e.g., Vickery et al., 2003; Gavanski and Kopp, 2011) have considered damage accumulation, although these mechanisms are somewhat different to the observed response of nailed connections in wood-framed buildings. In fact, the non-linear response of nailed connections is similar in many ways to the elasto-plastic and piecewise linear load–displacement curves considered by Georgiou et al. (1988).

Fig. 1. Photograph of a global roof failure in Saskatchewan, Canada, on July, 4, 2010.

The objective here is to develop a model which captures the essential features of global roof failures of wood-frame, residential buildings under fluctuating, time-varying wind loads in order to quantify the effects of storm duration. Most residential structures in North America are made with this form of construction; however, light-framed, wood roofs are also common in many countries around the world. The intent of this model is to be as simple as possible since it is to be used in Monte-Carlo simulations of many long duration storms, and, thus, much less computationally intensive than, for example, finite element analyses (FEA) that are commonly used for design. As such, many simplifying assumptions are required, and, as with all such models, a loss of accuracy is exchanged for simplicity. Thus, model validation is critical and is central to the current paper. Then, using the concept of the "design cyclone" (Jancauskas et al., 1994) together with the validated model, the effects of storm duration are examined.

2. Theoretical and analytical considerations

2.1. Underlying assumptions and simplifications

In the full-scale tests on a gable roof house at the "Three Little Pigs" project, Morrison (2010) and Morrison et al. (2012) made several observations pertaining to the response of the building under spatially and temporally-varying wind loads applied to the roof. In particular, they found that the largest displacements were at the toe-nailed, roof-to-wall connections (RTWCs). In addition, they observed that (i) the roof trusses responded to the uplifting wind load by exhibiting an approximate solid body rotation, but that (ii) each roof truss had distinct displacements at the connection to the top plate. In addition, the top plate and inter-storey displacements were found to be an order of magnitude smaller than those of the RTWCs. Based on this; we will only consider the withdrawal failures of the RTWCs, but not the top plates or inter-storey uplift failures. The implication of points (i) and (ii) above is that the truss bending stiffness is substantially higher than the bending stiffness of the roof in the direction normal to the plane of the trusses and the stiffness of the toe-nailed connections. This is, perhaps, not surprising, since the bending stiffness of the roof in the direction normal to the plane of the trusses is set by the bending stiffness of the roof sheathing, the fascia beams, gable end wall bracing, ceiling drywall, etc.

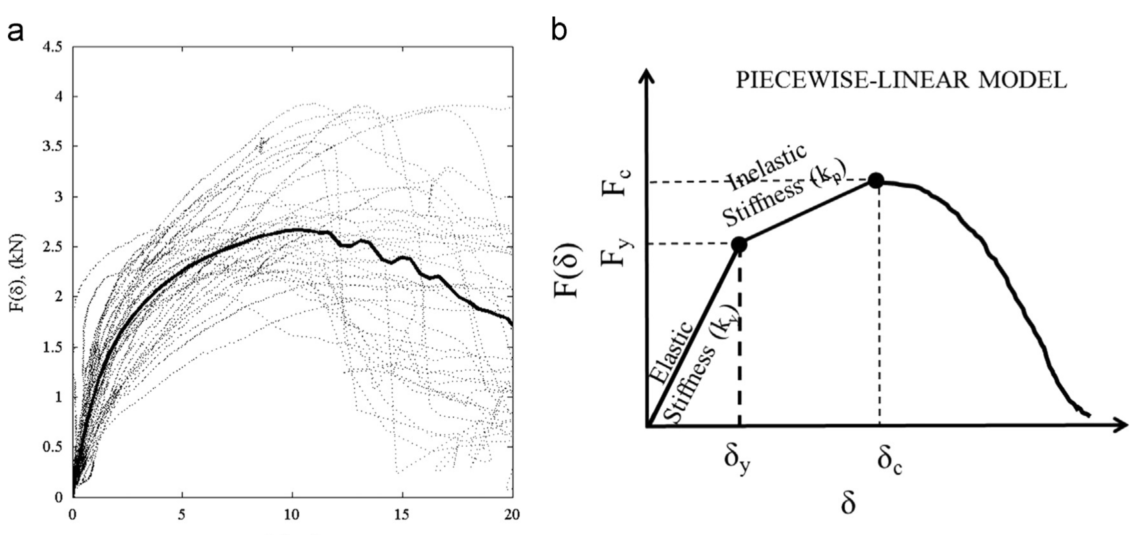

Fig. 2. (a) Load–displacement curves of 35 toe-nailed connections obtained by Khan (2012) along with the estimated mean load–displacement curve (in bold), with (b) a piecewise linear model of the load–displacement behavior.