INFLUENCE OF LIGHT-FRAME WOOD STRUCTURE PROPERTIES ON SEISMIC LOSS ESTIMATION

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ABSTRACT :

Light frame wood structures represent the vast majority of construction type for residential structures throughout North America. As a result of this large stock of buildings in the U.S., earthquake-induced losses for this type of structure could have a severe financial impact on both individuals and the community as a whole. The 1994 Northridge earthquake in California resulted in economic losses of more than $20 billion for wood frame buildings alone, and provides the impetus for this study. A comprehensive loss estimation procedure was combined with nonlinear time history analysis for light-frame wood structures to investigate the most influential sources of loss such as structural and non-structural damage and contents damage. The strength and stiffness of the structure was correlated with the change in the nail schedule for the shearwalls as well as construction quality. The effect of these variants on the short and long term financial loss was then investigated through loss simulations which utilize assembly-based vulnerability. With the help of automated structural dynamics and loss analysis package developed for woodframe structures (SAPWood) at CSU, seismic loss estimation for a typical Western style single family home building was conducted for a series of configuration variations. It was concluded that there exists an intensity sensitive region for strength and stiffness which limits the effectiveness of improvements for small or large earthquakes.

KEYWORDS: Seismic loss estimation, Woodframe structure, SAPWood, Sensitivity

1. INTRODUCTION

Light-frame wood (woodframe) buildings represent the main type of residential construction in North America. They have performed satisfactorily during earthquakes from a life-safety standpoint but can be quite vulnerable to damage and subsequent losses. More than half of the estimated $40 billion loss from the 1994 Northridge earthquake was the result of damage to woodframe buildings. Although collapse is rare for code-designed woodframe buildings, the damage sustained by these structures and the cost to repair them following an earthquake can result in financial ruin for home owners without adequate earthquake insurance. As the seismic research community began to investigate loss related issues and their societal impact, studies and tools to estimate loss on a large (regional) scale (e.g. HAZUS program from the Federal Emergency Management Agency) was developed to help with the decision-making process. Detailed studies related to specific building types were also conducted for steel (Liu et al., 2003) and concrete buildings (Ang et al., 2001). Only limited research (Porter et al., 2001) related to losses for woodframe structures have been performed. This is likely due to the complexity of the nonlinear dynamic response, the relatively low cost of a single woodframe building, and the significant contribution to response and loss by the non-structural components.

Although it is apparent from experience that higher seismic hazards and weaker structures will typically result in larger losses over time, the quantitative relationship between loss and the structural design and configuration parameters has not been studied thoroughly for wood frame structures. The assembly based vulnerability (ABV) framework proposed by Porter (2000) provides a quantitative procedure to estimate losses for a woodframe structure from earthquakes. This is done by summing damage and costs from individual damageable components based on nonlinear time history results. Pei and van de Lindt (2008) adopted the ABV based method in their development of a vulnerability model and applied it to develop a long term loss simulation framework. The entire procedure is incorporated into the newly developed software package Seismic Analysis Program for Woodframe Structures (SAPWood). With these available methods and tools, this study focused on quantitatively investigating the influence of structural properties on earthquake-induced losses to woodframe buildings.

2. LOSS ESTIMATION PROCEDURE

Examining earthquake induced losses to woodframe buildings requires a comprehensive loss estimation procedure to assess losses for a variety of structural configurations under earthquake hazard. In this study, the financial loss was calculated using the Monte-Carlo simulation based procedure developed by Pei and van de Lindt. A summary of the procedure is presented in Figure 1.

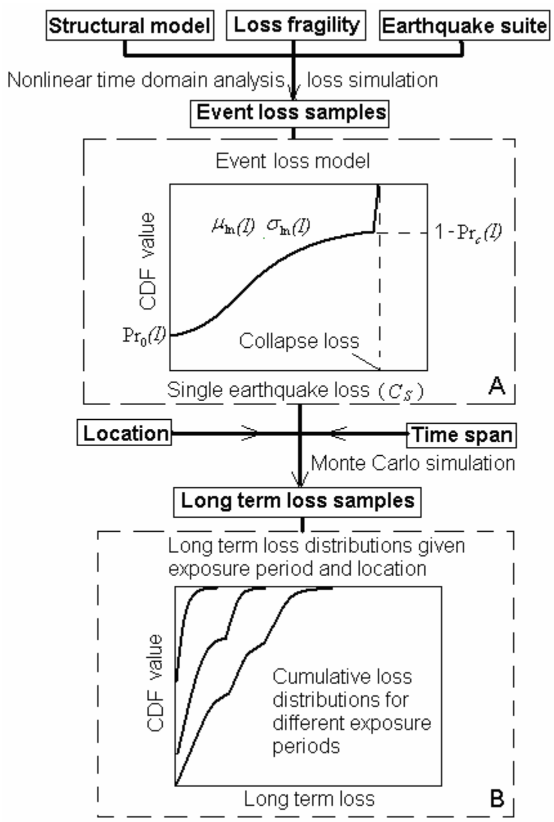

Figure 1 Loss estimation framework

The procedure mainly consists of two simulation steps. The first simulation step focuses on single earthquake event loss, which is the loss of a given building due to a single earthquake event. This simulation procedure produced loss samples from nonlinear time history analysis and represented them with statistical distributions. Then the relationship between distribution parameters and seismic intensity was established (either in explicit functional form or in an empirical way) and termed herein as a vulnerability model. This model serves as a comprehensive probabilistic representation for the loss resistance of the structure against a single earthquake event. If the cumulative loss over a certain exposure period is of interest, the second step of the simulation procedure termed herein as long term loss simulation will be performed. The objective then becomes to obtain the distribution model for long term loss which is illustrated in block B of Figure 1. In this procedure, the vulnerability model is combined with uncertainty models for earthquake occurrence and intensity. The earthquakes occurred in the given exposure period were generated based on historical data from the U.S. Geological Survey (USGS) database. The loss from each event was generated from the vulnerability model and the cumulative loss was then found by adding the individual losses together.

3. STRUCTURAL PROPERTIES

Shearwalls are the main lateral force resisting component in light frame wood structures, and are the focus of the design procedure for strength based seismic design. The nailing pattern and sheathing panel thickness (and type) are determined based on the amount of lateral force needed for each individual wall line. Different wall configurations, when subjected to seismic excitation, can lead to very different values of seismically-induced loss in a woodframe building. A quantitative understanding on the sensitivity of loss behavior to changes in structural properties is crucial in order to develop a better understanding of how these design variants should enter into the loss-based decision making and design process. In order to investigate this sensitivity, quasi-static numerical models for different shearwalls were established in the SAPWood-Nail Pattern (Pei and van de Lindt, 2007) software package and forced through a reversed-cyclic displacement protocol imposed at the top of the wall. The resisting shear force of the entire wall during the loading process can be calculated based on the principal of virtual work and the hysteretic response of the wall is obtained. Each individual wall hysteresis is fit to a wall level nonlinear hysteretic spring model (see Pei and van de Lindt, 2007 for details on the model), which is then put into a full system-level model (i.e. a house) in order to perform nonlinear time history analyses to obtain responses for loss estimation. Through this procedure, the difference in shearwall design (properties) is represented in loss simulation as shearwall hysteresis elements having different parameters.

Nail Pattern/Schedule and construction quality

Three types of very commonly used nailing patterns (in the U.S.) were investigated, termed as 2/12 (sheathing panel edge nails spaced at 50mm with field nailing at 305mm), 4/12 (edge spacing = 102mm, field spacing =305mm), and 6/12 (edge spacing = 152mm, field spacing = 305mm) nailing pattern. The standard wall model was selected as a 2.44 x 4.88 m (4 x 8 ft) shear wall with 0.4 m stud spacing. The sheathing to framing nails used in the model were 8d common nails (length = 63.5mm, diameter = 3.33mm) which is also widely used in residential light frame wood construction. Fastener parameters were obtained through cyclic tests of fasteners with 11.1mm thick (7/16 inch) OSB sheathing conducted at Colorado State University. Since wood shearwall behavior is largely controlled by the backbone curve of the hysteresis, the backbone curve parameters obtained from the analysis are presented in Table 1. Other factors that might affect the overall structural and shearwall performance, such as construction quality are also shown in that table. An earlier study (Kim and Rosowsky, 2002) indicated that construction quality issues in light frame construction usually result from poor on-site construction practice, such as sheathing nails missing the wall stud and thus reducing the ultimate capacity and stiffness of the wall and essentially changing its behavior during an earthquake. In this study, the shearwall models corresponding to variants of poor construction practices were modeled with SAPWood NP, which included missing 20% of the nails, missing one field nail line, and missing one panel edge nail line (vertical nail line, top nail line, and bottom nail line). The locations of these missing fasteners are presented in the schematic of Figure 2. The wall models for these cases were built and analyzed for each configuration. Also shown in Figure 2 are the backbone curves corresponding to the model with construction quality deficiencies, and an ideal wall model (no missing fasteners) backbone for reference.

As one might anticipate, it can be concluded from Table 1 that both the variation in nail pattern and construction quality result in a “degradation” (or reduction) of the hysteretic parameters, especially those associated with the strength and stiffness of the nonlinear hysteretic spring. Using the parameters for an ideally built 2/12 wall as a benchmark value (full capacity) against which to compare other models with construction deficiencies, the degradation of stiffness and strength parameters under all situations in Table 1 can be illustrated with percent reductions to the key hysteretic parameters (K0 and F0) in Figure 3.

Table 1. Wall model backbone parameters (N-mm)

Pattern	Quality	k0	f0	r1	xu	r2	xu1	λ	fur
2/12	Ideal	2851	33569	0.01	32	-0.05	38	-1.00	374
	Missing Field nail	2441	33302	0.01	35	-0.05	42	-1.00	356
	Missing top line	2364	31581	0.01	30	-0.05	37	-1.00	334
	Missing bottom line	2277	30700	0.01	30	-0.05	37	-1.00	320
	Missing 20% overall	2101	29077	0.01	26	-0.05	32	-1.00	276
	Missing edge line	1692	15879	0.01	31	-0.05	38	-0.60	160
4/12	Ideal	2087	17752	0.01	27	-0.05	32	-0.70	196
	Missing Field nail	1786	16987	0.01	26	-0.05	31	-0.80	187
	Missing top line	1769	15879	0.01	29	-0.05	35	-0.70	178
	Missing bottom line	1786	15074	0.01	28	-0.05	34	-0.70	169
	Missing 20% overall	1664	14216	0.01	27	-0.05	32	-0.50	151
	Missing edge line	1191	9430	0.01	30	-0.05	36	-0.70	107
6/12	Ideal	1721	11632	0.01	23	-0.05	28	-1.00	129
	Missing Field nail	1646	11053	0.01	23	-0.05	28	-0.70	125
	Missing top line	1541	10213	0.01	25	-0.05	30	-0.70	116
	Missing bottom line	1471	10577	0.01	26	-0.05	31	-0.70	116
	Missing 20% overall	1366	9216	0.01	25	-0.05	30	-0.60	102
	Missing edge line	1051	6685	0.01	26	-0.05	32	-0.60	76

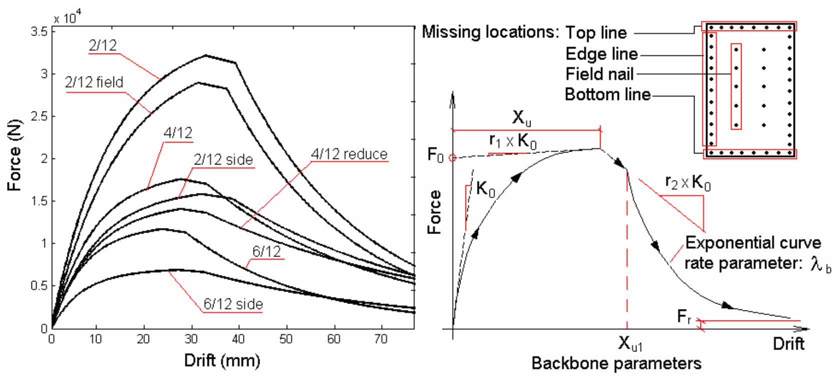

Figure 2 Degrading of backbone curve for different wall configurations

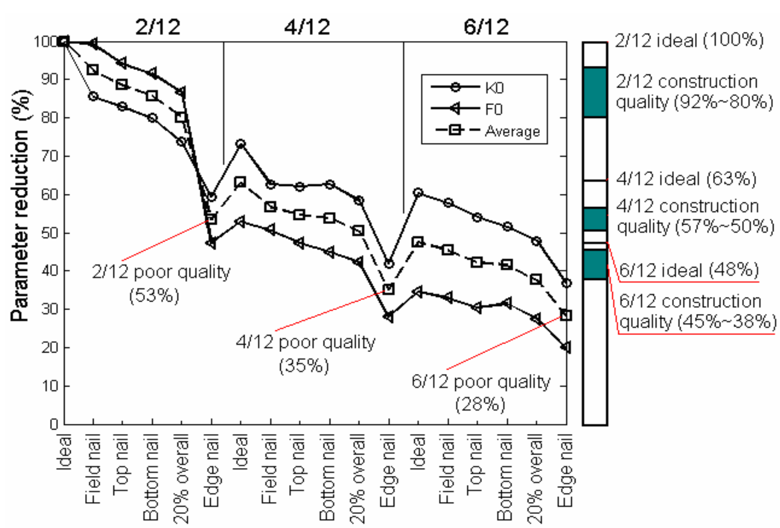

Figure 3 Quantifying nailing pattern and quality with parameter reduction

4. EXAMPLE STRUCTURE

In order to examine the sensitivity of loss to structural parameters of residential structures, a two story single family home with two bedrooms and a one car garage was selected in this paper as the example structure. This type of residential building was felt to represent the dominant residential construction type in North America and is felt to be appropriate for a seismic loss study. The building has a total area of about 140 m2 (1500 sq. ft), and the architectural floor plan is shown in Figure 4.

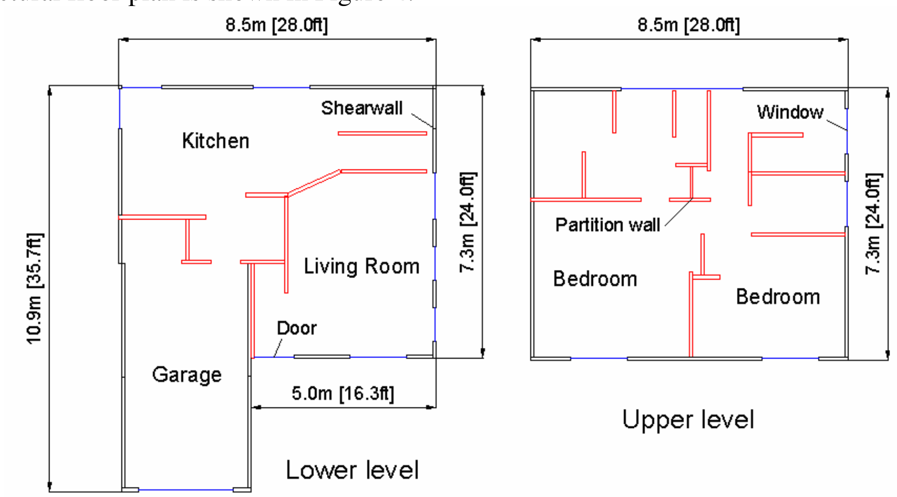

Figure 4 Example building floor plan

The numerical model for the structure was developed using the software program SAPWood (Pei and van de Lindt, 2007) with shearwalls represented by nonlinear hysteretic elements as described earlier. This level of numerical model complexity is consistent with typical nonlinear analysis within the seismic wood engineering research community. The seismic mass was assumed to be evenly distributed at the first floor level (19,500 kg [43 kips]) and the roof level (15000 kg [33 kips]). Four types of damageable components were considered in the loss estimation, which included structural shearwalls, drywall partition walls, doors and windows, and general contents. The repair cost of each component was based on the structural response from nonlinear time history analysis and the component damage fragilities (Pei and van de Lindt, 2008). Note that the inclusion of components contributing to the loss in this study was not comprehensive, i.e. the collapse loss value is likely to be smaller than financial loss in a real collapse since only four major components were included. But after the simulated losses were normalized by the collapse loss value, the results and conclusions based on the normalized values was felt to be representative for typical residential woodframe building.

5. LOSS SENSITIVITY

The structural parameter variation will have a direct impact on the vulnerability analysis and result in different loss behavior for a single earthquake event. A suite of 20 earthquake ground motion records (Krawinkler, 2002) was used in the vulnerability analysis to represent the variation in earthquake ground motion. Based on the relationship between wall element parameters and structural configuration/quality illustrated earlier in Figure 3, the change in nailing pattern and different levels of construction quality were examined here by incrementally reducing the stiffness and strength parameters of all wall elements in the numerical model with a reduction factor (discount) from 0% to 70% of the full value. A 0% reduction factor corresponds to an ideally built wall with the 2/12 nailing pattern without any construction quality deficiencies.

Following the aforementioned vulnerability simulation procedure, the vulnerability parameters for the example house with different reduction factors were calculated and plot as vulnerability curves in Figure 5.

Note that the seismic intensity was characterized by spectral acceleration at a period of 0.2 seconds with a 5% elastic damping ratio. The 0%, 35%, and 50% reduction cases correspond to ideally built walls with the 2/12, 4/12, and 6/12 nailing patterns respectively. The 70% reduction case corresponds to a wall having a 6/12 nailing pattern with poor construction quality, which was the worst case included in this study. A direct comparison of mean loss values were also compared among alternative options in Figure 6, where the average loss of the 2/12 ideal case was shown in (d) while the difference in the average event loss between several of the reduced cases and the 2/12 ideal case are presented in (a), (b), and (c).

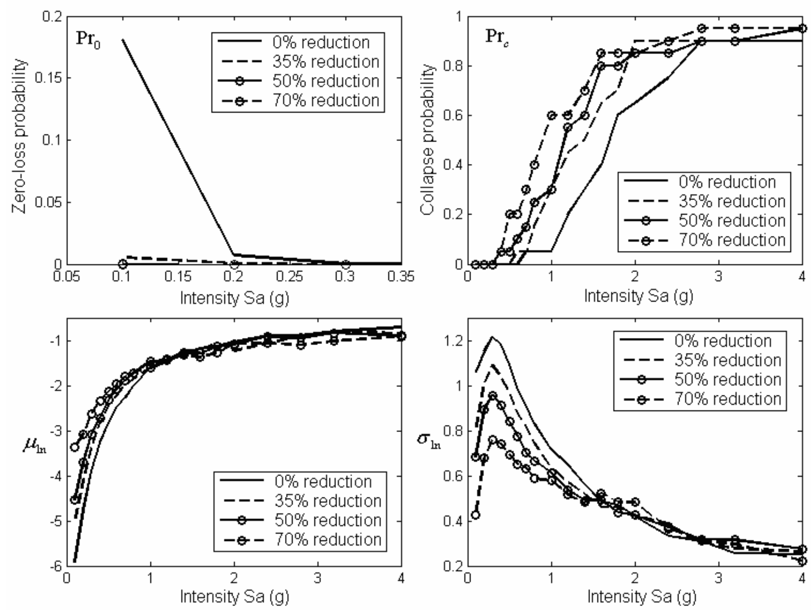

Figure 5 Effects of reduction factor on vulnerability parameters

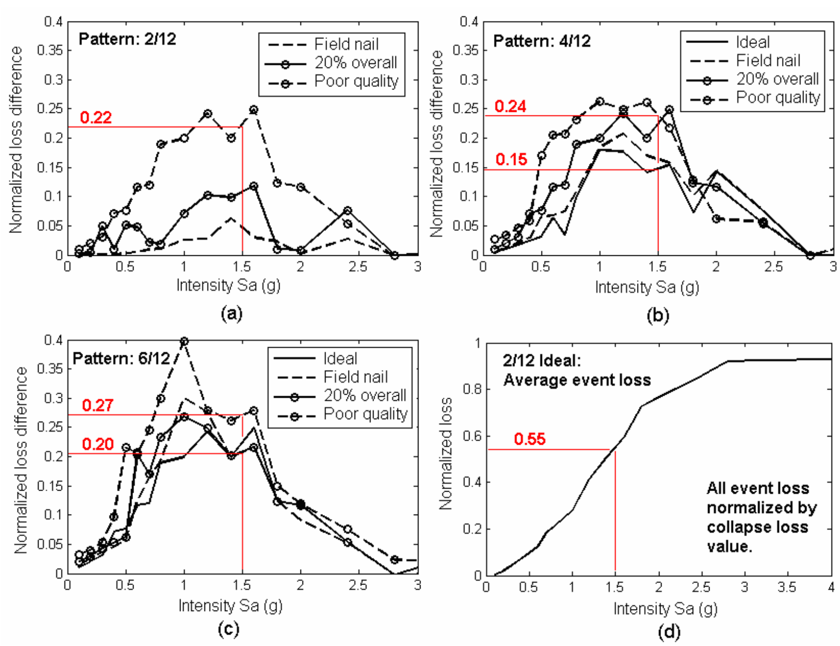

Figure 6 The impact of configuration/quality issue on average event loss

It can be observed from Figure 6 that the financial loss of the building with the stronger wall configuration is generally less than that with weaker walls provided the construction quality is the same, which is to be expected for the same seismic mass. The impact of construction quality deficiencies is more notable for stronger configurations. For example, as one can see from Figure 6, given an earthquake with 1.5 g spectral acceleration, the average single earthquake event loss for the 2/12 nailing pattern structure could range from 55% to 77% (a 22% variation) of the collapse loss depending on the construction quality. The range of average loss was 70% to 79% (a 9% variation) for the 4/12 pattern and 75% to 82% (only a 7% variation) for the 6/12 pattern. The benefit observed from quality control is more significant for stronger designs. Among different seismic intensity levels, the difference in average loss was not significant in both the low and high ends of the intensity level. The simulation showed essentially no difference in average loss for intensities over 3g spectral acceleration due to the fact that the model indicated collapse for most simulation cases regardless of quality or nailing pattern. This implies that the financial advantage of having a stronger building or better quality construction would not be significant if the earthquake is very small or very large, but is present for moderate seismic intensity levels. In the case of this example, the range of (approximately) 0.3 g to 2.5 g spectral acceleration was the intensity level within which the structural configuration makes a significant difference. This range was termed herein as the Intensity Sensitive Region (ISR) for structural properties.

Long term loss simulation was also performed for the structural variations using the hazard data for Los Angeles, CA. The statistics of the simulated loss distribution are presented in Table 3, including the probability of zero loss (Pr0), the median, and the 95th percentile value (labeled Extreme). The "Normal" quality in the table corresponds to the case with 20% nails either missing or missing the stud, representing the most deficient construction quality among normal quality problems. The "Poor" quality represents missing the edge nail line which induces the maximum strength reduction.

Table 2. Normalized* long term loss

Nailing pattern	Period (year)	Ideal			Normal			Poor		
		Pr0	Median	Extreme	Pr0	Median	Extreme	Pr0	Median	Extreme
2/12	5	0.23	0.01	0.11	0.20	0.01	0.13	0.18	0.02	0.20
	30	0.00	0.09	0.56	0.00	0.12	1.02	0.00	0.23	1.17
	75	0.01	0.28	1.20	0.01	0.38	1.47	0.01	0.69	1.78
4/12	5	0.18	0.01	0.17	0.18	0.02	0.20	0.18	0.05	0.24
	30	0.00	0.16	1.04	0.00	0.23	1.17	0.00	0.41	1.28
	75	0.01	0.48	1.58	0.01	0.69	1.78	0.01	1.07	2.40
6/12	5	0.18	0.02	0.20	0.18	0.04	0.21	0.18	0.06	0.27
	30	0.00	0.23	1.17	0.00	0.33	1.18	0.00	0.47	1.41
	75	0.01	0.69	1.78	0.01	0.93	2.02	0.01	1.33	2.49

* Long term loss was normalized by collapse loss of the building, which represent the total loss caused by complete collapse of the structure.

6. CONCLUSIONS

Based on detailed nonlinear dynamic model responses, vulnerability analysis was conducted for a typical North American style residential building with variants including structural properties and construction quality. Earthquake induced loss was assumed to be a random variable whose distribution was controlled by the structural and seismic inputs. A quantitative relationship between construction quality, structural strength and stiffness, and the expected earthquake induced loss was established. Through the examination of the results, an Intensity Sensitive Region associated with the modification of structural strength and stiffness was identified. As a direct result, the mitigation of loss due to changes in structural properties or construction quality is quite limited for very small or very large earthquakes. Within the Intensity Sensitive Region, the impact of construction quality on event loss is more significant in stronger configurations than in weaker ones. However, using a significantly stronger structural configuration could considerably reduce the expected losses in seismically active regions. The shear wall configuration and construction quality had a considerable impact on short term and long term seismic losses.

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