COMPARISON OF SEISMIC FRAGILITY OF FREE STANDING EQUIPMENT USING CURRENT TESTING PROTOCOLS AND RECORDED BUILDING FLOOR MOTIONS

Clayton Burningham
University of Utah
Gilberto Mosqueda, PhD
Rodrigo Retamales Saavedra
University at Buffalo, State University of New York

Abstract

Present structural design is concerned with life protection and collapse prevention of a structure. The next generation of design codes is poised to introduce function protection for critical buildings (such as hospitals), for which protecting nonstructural components and building contents is a priority. This priority is due to the fact that the majority cost of a structure lies in nonstructural components and building contents such as partition walls, suspended ceilings, furnishings, equipment (electrical and mechanical), distributed lines, etc. As a result, even though a structure avoids catastrophic collapse, damage repair and replacement costs of nonstructural components generate a significant expense for building owners. This means that nonstructural components are the most important factor in seismic losses. Additionally, past earthquakes have demonstrated that for essential or priority buildings (such as hospitals), although the structure remains standing, damage to nonstructural components renders the facility inoperable, and therefore useless to the community around it during the moment of largest need for health services.

It is therefore important to evaluate the fragility of nonstructural components. Some protocols for evaluating the fragility of nonstructural components already exist, and they have been developed for the seismic demands generally imposed upon linear and slightly nonlinear systems of single and multiple degrees of freedom. This paper presents an analysis and comparison of seismic fragility of freestanding, bodies obtained through computer simulation using current testing protocols. It also determines which protocol is the most conservative for conducting fragility analysis.

Introduction

Present structural design is concerned with life protection and collapse prevention of a structure. Over time, advances in structural engineering have created a large base in expertise that has conquered this problem. As a result, structures built with current methods are generally able to withstand seismic activity and maintain their structural integrity. However, although a building remains structurally sound, it can be rendered unusable due to nonstructural component damage. Additionally, the majority of the cost of a building lies in nonstructural components. It has been shown that nonstructural components account for 82%, 87% and 92% of building costs for offices, hotels and hospital buildings (Taghavi and Miranda, 2003). Therefore, although structural damage is avoided, building owners can incur great expense due to repair and replacement costs of nonstructural components.

It is also of note that building owners lose revenue during the time period in which their building is inoperable. Operability of critical structures such as hospitals is crucial during post-earthquake response of populated areas to disasters. Examples of damage to critical structures and its catastrophic effects on communities were seen in the 1999 Chi-Chi Earthquake (Soong, 2000). Additional damage to critical structures was seen in the 1994 Northridge earthquake, the 2001 El Salvador earthquake, and the 2006 Hawaii earthquake (Mosqueda, et al. 2007).

It is clear to see that nonstructural components play an important role in the cost and operability of a building. It is therefore prudent to minimize damage to nonstructural components during seismic activity. Additionally, as engineering capabilities to minimize nonstructural component damage advance, building owners will have more control over building performance as it relates to nonstructural components (Krawinkler, 1999). A necessary component of this Performance Based Engineering is fragility analysis of nonstructural components, which is simply the probability of meeting a specified damage state for a given engineering demand parameter (EDP). By evaluating the fragility of individual components, more accurate predictions of nonstructural component behavior under seismic activity can be made. These more accurate predictions in turn, serve to improve engineering, cost benefit, and safety.

Current Procedures for Fragility analysis and Seismic Qualification of Nonstructural Components

Two leading protocols used for seismic qualification and fragility analysis of nonstructural components are AC156 (ICC-ES, 2007) and FEMA461 (FEMA, 2006), respectively. It is also of note that these protocols are intended for separate evaluation of acceleration and displacement sensitive nonstructural components. In other words, systems of nonstructural components cannot be evaluated using either of the abovementioned protocols.

AC156 was developed by the International Code Council Evaluation Service in order to provide guidelines for compliance with building codes, such as the International Building Code which requires seismic qualification of nonstructural components and equipment used in critical facilities. The protocol is specifically intended for shake table qualification of nonstructural components and equipment. Input acceleration time histories are of 30 second duration, range between a frequency of 1.3 to 33.3 Hz, and assume a damping value of 5% critical damping.

FEMA461 is the most recent protocol created for experimental research. It was proposed as a set of procedures to establish “seismic performance characteristics” of nonstructural components (FEMA 2006). Additionally, it was developed in conjunction with the Applied Technology Council, the Mid America Earthquake Engineering Research Center (MAE Center), the Multidisciplinary Center for Earthquake Engineering Research (MCEER), and the Pacific Earthquake Engineering Research Center (PEER), with funding from the Federal Emergency Management Agency and the National Science Foundation. Its primary purpose is performance evaluation of nonstructural components through fragility functions. Although its main purpose is not for seismic qualification of nonstructural components, the protocol can be used to do so (FEMA 2006). It is also of note that FEMA461 contains provisions for both quasi-static cyclic testing and shake table testing of displacement and acceleration sensitive nonstructural components, respectively.

Although AC156 and FEMA461 testing protocols are defined for seismic qualification and fragility analysis of nonstructural components respectively, it is unknown which more accurately can be used in evaluating the seismic fragility of nonstructural components and equipment, when compared to the effects of real building floor motions. Therefore, it is necessary to compare fragility curves generated based on the two protocols to fragility curves generated based on actual building acceleration histories.

Experiment Set-up

An analysis of fragility curves generated in accordance with the abovementioned protocols and real building histories was accomplished through computer simulation. Working Model 2D software was implemented to create a “virtual” shake table. Compatible time histories for AC156 and FEMA461 and real building time histories were used as input for simulation. Figure 1 shows two input motions compatible with FEMA461 used in the computer simulation.

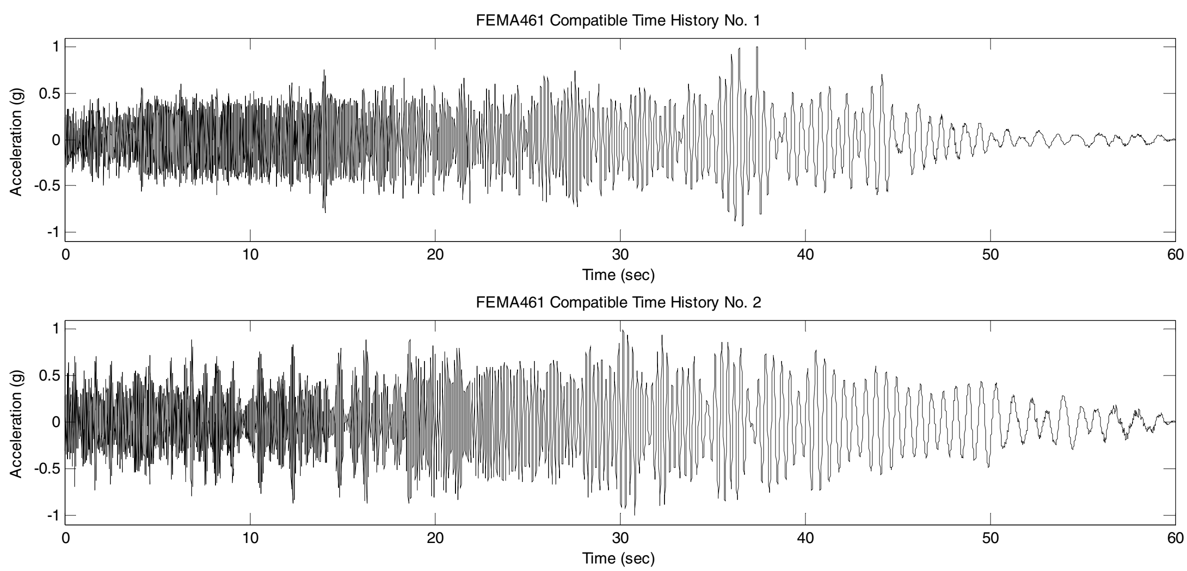

Figure 1. FEMA461 compatible time histories

Similarly, two input motions compatible with AC156 used in the simulations can be seen in Figure 2. The FEMA461 and the AC156 time histories were scaled to a peak acceleration value of 1g. In order to choose a set of real building time histories for use in the analysis, six buildings were selected from the California Strong Motion Instrumentation Program (CISMIP) database, and were required to meet the following criteria:

- Located less than 15 km from the epicenter of the earthquake for which the building response was recorded.
- Representative of different structural earthquake resistant systems (concrete shear walls, moment resistant steel and concrete frames, and mixed resistant systems)
- Containing 4 to 54 floors
- Subjected to several large scale earthquakes

A further description of the selected buildings can be found in Nonstructural Components and the Rainflow Counting Algorithm (Kelleher, 2007).

Each sensor of record in the group of six selected buildings was assigned a number. To ensure an unbiased sampling of sensor histories within the representative group of buildings, a random number generator was implemented to narrow the real building time histories to ten sensor histories. As a result, the real building histories accounted for not only ground motions, but also floor motions at various stories in buildings of varying fundamental periods. The sensors selected for use in generating the fragility curves based on real building histories can be seen in Table 1. All real building floor motion histories were scaled to a peak acceleration of 1g.

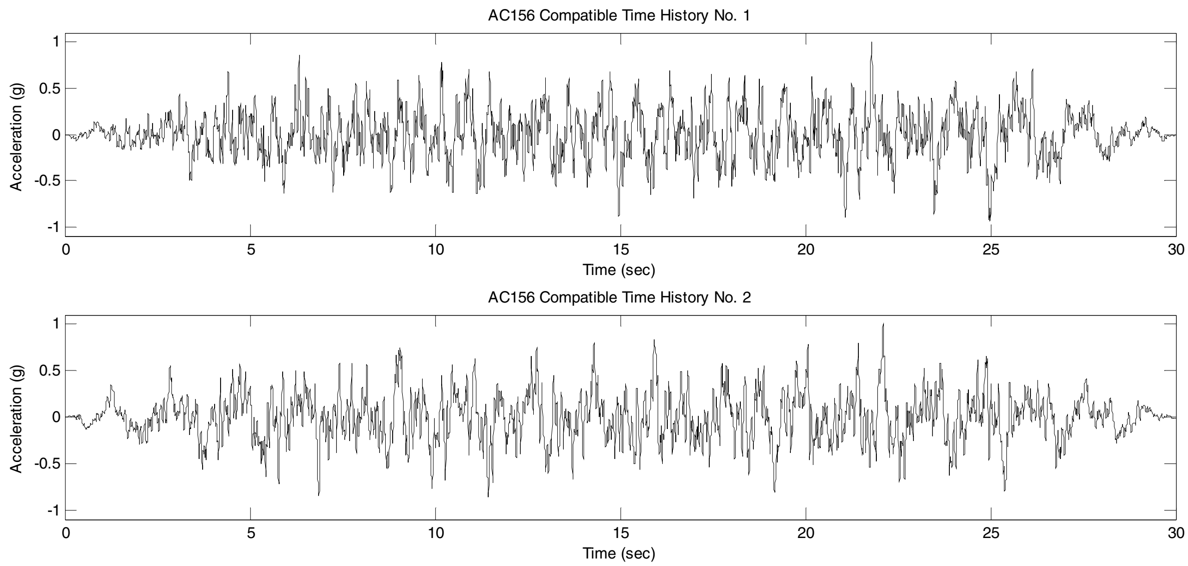

Figure 2. AC156 compatible time histories

Table 1. Buildings and sensors used in analysis

Number	Details
1	Building 24322 - Sensor 8
2	Building 24322 - Sensor 6
3	Building 24514 - Sensor 5
4	Building 24514 - Sensor 9
5	Building 24514 - Sensor 8
6	Building 24629 - Sensor 7
7	Building 47459 - Sensor 8
8	Building 24386 - Sensor 11
9	Building 24514 - Sensor 13
10	Building 24464 - Sensor 8

For the analysis, self-standing nonstructural components were modeled as freestanding rigid bodies or blocks. This model was representative of typical non-anchored nonstructural components and building contents such as computer monitors and CPU's, bookshelves, filing cabinets, medical imaging equipment, chairs, microscopes, etc. In order to account for different geometries in the abovementioned typical nonstructural components, several base-to-height ratios were considered in the modeling. These considered base-to-height (slenderness) ratios were 0.33, 1.00, and 2.00. All blocks were modeled as having a constant density of 1 gm/cm^3. Additionally, coefficients of static and kinetic friction were calculated as an average of those found by Chaudhuri and Hutchinson for equipment placed on ceramic laboratory benches (2004). The calculated averages for the coefficient of kinetic friction and coefficient of static friction on were 0.59 and 0.53, respectively. Also, the coefficient of restitution (the ratio of pre- and post-impact velocities) was held constant at 0.5 for all bodies in the simulation.

Virtual shake table tests were run on blocks of varying heights with the same base-to-height ratio for each of the 14 input time histories. Dynamic response history data collected included the horizontal translation of the blocks relative to the shake table as well as the angle of rotation of the blocks. Four damage states were considered. For rigid bodies with a base-to-height ratio of 0.33, the only damage state considered was block overturn (+/- 90 degrees of rotation), given that in all testing cases the block overturns before initiating sliding. For rigid bodies with base-to-height ratios of 1.00 and 2.00, the damage states were selected in accordance with work conducted by Soong et al. (2000) regarding sliding fragility of freestanding rigid body nonstructural components. Correspondingly, the considered damage states were maximum relative horizontal displacement of 25 mm (1 in), 50 mm (2 in), and 75 mm (3in). Based on these damage states (DS), fragility curves were developed based on Equation 1 (FEMA, 2006).

$$f(DP) = P[D \geq DS \mid EDP] \qquad \text{(Eq. 1)}$$

where D is the damage sustained by the component. DS is a specific damage state, and EDP is the engineering demand parameter. This function is essentially the probability that the damage state DS is met for a given Engineering Demand Parameter (EDP). Fragility curves are “idealized by a lognormal distribution” because of its ability to portray the trends of “structural failure data” (Porter et al. 2007). The EDP considered in this analysis was the peak shake table acceleration.

Results

The resulting fragility curves for blocks with a base-to-height ratio of 0.33 can be seen in Figure 3. As mentioned before, the key damage state for blocks with this base-to-height ratio was that of block overturn.

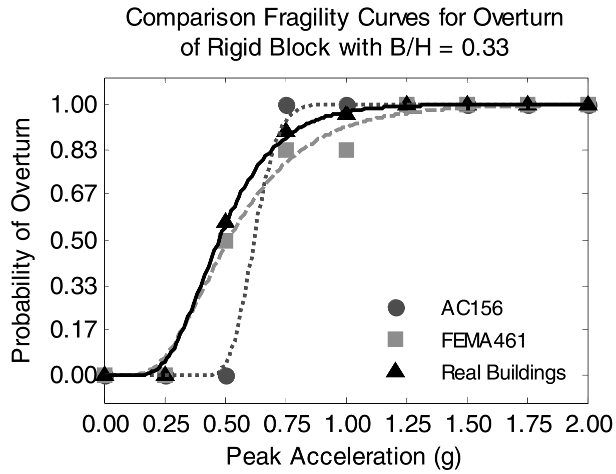

Figure 3. Comparison fragility curves for several testing protocols. Rocking block B/H = 0.33

Analysis of the curves in Figure 3 shows that in the case of a base-to-height ratio of 0.33, the fragility curve based on real building motions is most closely resembled by the curve based on the time history in accordance with FEMA461. The fragility curve obtained using the FEMA461 protocol has a standard deviation close to that observed for the simulation performed using recorded floor motions. The values for mean acceleration required to induce block overturn in Table 2 show that the curves based on AC156 and FEMA461 underestimate the mean peak acceleration required for block overturn. However, it is important to note that the protocols overestimate the median acceleration required to induce block overturn! This means that the AC156 and FEMA461 protocols are conservative with respect to median acceleration required to meet the rocking damage state, with AC156 being the most conservative as evidenced by a median acceleration percent difference of 26.5%.

Table 2. Mean and median accelerations and percent difference for block with B/H = 0.33

Acceleration History	DS = Block Overturn			
	Mean Acceleration (g)	Percent Difference (%)	Median Acceleration (g)	Percent Difference (%)
AC156	0.6268	19.7	0.6221	26.5
FEMA461	0.5798	11.9	0.5163	8.0
Real Building Histories	0.5145	0.0	0.4766	0.0

The fragility curves generated for blocks with a base-to-height ratio of 1.00 and 2.00 can be seen in Figures 4 and 5, respectively. As mentioned previously, the key damage states considered for blocks with a base-to-height ratio of 1.00 and 2.00 were those of 25 mm (1 in), 50 mm (2 in), and 75 mm (3 in) of relative slide. Analysis of Figures 4 and 5 shows that when considering the damage state of 25 mm of slide, curves based on AC156 and FEMA461 are the same yet underestimate the mean peak acceleration required for the block to meet the 25 mm slide damage state.

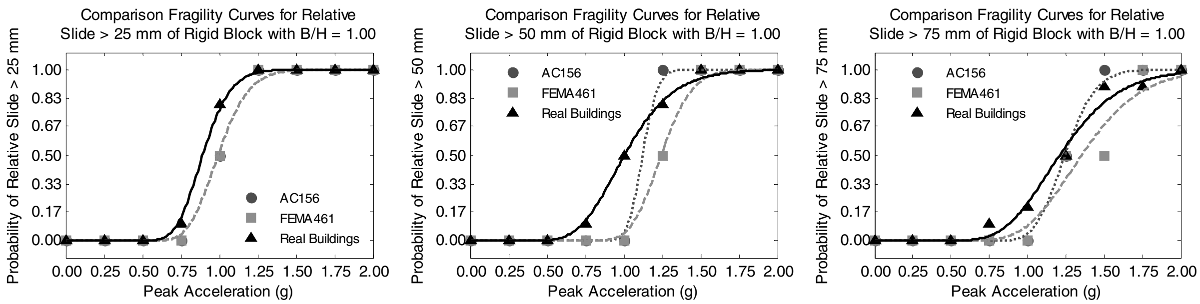

Figure 4. Comparison fragility curves for several testing protocols. Sliding block B/H = 1.00

It is of note, however, that the standard deviation for all three curves in Figures 4 and 5 is the same for the 25 mm slide damage state. In addition, a comparison of the curves for the damage states of 50 mm and 75 mm of relative slide in Figure 4 and 5 show that the shape of the real building results are best approximated by FEMA461. As with the curves in Figure 3, the curves generated from the protocol compatible time histories underestimate the mean acceleration required to meet all of the relative slide damage states. Numerical values for mean acceleration required to meet the sliding damage states for a block with a base-to-height ratio of 1.00 and 2.00 can be seen in Tables 3 and 4, respectively.

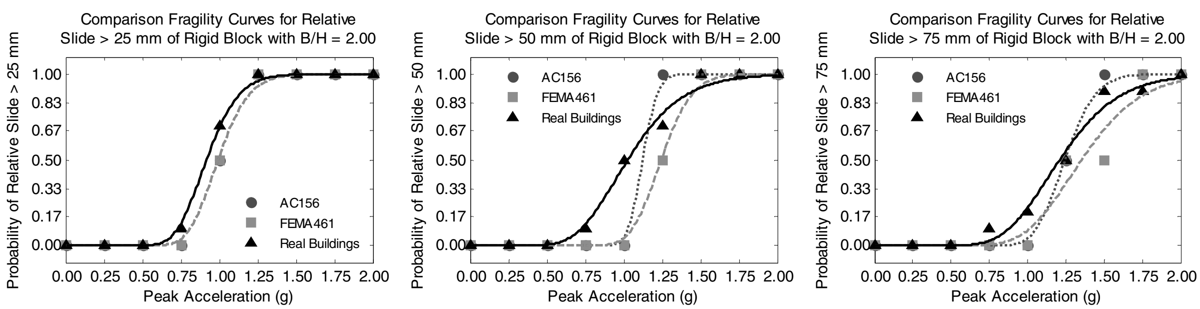

Figure 5. Comparison fragility curves for several testing protocols. Rocking block B/H = 2.00

Further comparison of the data seen in Table 3 shows that both AC156 and FEMA461 make a conservative estimation of the median acceleration required to reach all the relative slide damage states with a probability of 50%. It is also of note that the percent difference values are greater for the results from the FEMA461 curves than for the results from the AC156 curves. Thus, for a base-to-height ratio of 1.00, FEMA461 is the more conservative and, consequently, the safest protocol of the two when comparing the median acceleration required to meet sliding damage states.

Table 3. Mean and median accelerations and percent difference for block with B/H = 1.00

Acceleration History	DS = 25 mm Relative Slide			
	Mean Acceleration (g)	Percent Difference (%)	Median Acceleration (g)	Percent Difference (%)
AC156	1.0052	10.6	0.9927	10.6
FEMA461	1.0052	10.6	0.9927	10.6
Real Building Histories	0.9036	0.0	0.8924	0.0

Acceleration History	DS = 50 mm Relative Slide			
	Mean Acceleration (g)	Percent Difference (%)	Median Acceleration (g)	Percent Difference (%)
AC156	1.1260	8.3	1.1234	11.1
FEMA461	1.2541	19.0	1.2441	21.2
Real Building Histories	1.0360	0.0	1.0052	0.0

Acceleration History	DS = 75 mm Relative Slide			
	Mean Acceleration (g)	Percent Difference (%)	Median Acceleration (g)	Percent Difference (%)
AC156	1.2541	0.5	1.2441	2.8
FEMA461	1.3878	10.6	1.3534	11.2
Real Building Histories	1.2478	0.0	1.2101	0.0

Similar trends can be seen in the fragility curves for blocks with a base-to-height ratio of 2.00 visible in Figure 5. Visual comparison of the curves yields the same conclusions as those from comparing the curves for the 1.00 base-to-height ratio except that for the 25 mm slide damage state, both AC156 and FEMA461 show an improvement in ability to approximate the shape of the curve generated by real building histories. Additionally, it can be seen from the values in Table 4 that at a base-to-height of 2.00, the results from FEMA 461 are overestimates of the median acceleration required to reach all the relative slide damage states with a probability of 50% as evidenced by the percent differences for the median accelerations. Also, both protocols underestimate the mean acceleration required to meet all of the relative slide damage states at a base-to-height ratio of 2.00.

Table 4. Mean and median accelerations and percent difference for block with B/H = 2.00

Acceleration History	DS = 25 mm Relative Slide			
	Mean Acceleration (g)	Percent Difference (%)	Median Acceleration (g)	Percent Difference (%)
AC156	1.0052	7.6	0.9927	6.3
FEMA461	1.0052	7.6	0.9927	6.3
Real Building Histories	0.932	0.0	0.9173	1.6

Acceleration History	DS = 50 mm Relative Slide			
	Mean Acceleration (g)	Percent Difference (%)	Median Acceleration (g)	Percent Difference (%)
AC156	1.1260	5.7	1.1234	9.1
FEMA461	1.2541	16.4	1.2441	19.2
Real Building Histories	1.0640	0.0	1.0258	0.0

Acceleration History	DS = 75 mm Relative Slide			
	Mean Acceleration (g)	Percent Difference (%)	Median Acceleration (g)	Percent Difference (%)
AC156	1.2541	0.5	1.2441	2.8
FEMA461	1.3878	10.6	1.3534	11.2
Real Building Histories	1.2478	0.0	1.2101	0.0

Conclusions and Recommendations

Based on the aforementioned trends noted in the generated fragility curves, it is recommended that further analysis be conducted in order to determine the role of the static and kinetic coefficients of friction in fragility. It is anticipated that these parameters would significantly impact the fragility curves for freestanding rigid body nonstructural components. Additionally, it is recommended that a more extensive evaluation of performed on the relationship between base-to-height ratio and fragility. The close similarity in the results for a base-to-height ratio of 1.00 and 2.00 suggests that fragility depends on whether or not the base-to-height ratio is greater than or less than 1.00, and will not experience great changes within the two categories.

Overall, it was concluded that both AC156 and FEMA461 compatible time histories produce fragility curves that underestimate the mean acceleration required to meet a given damage state. However, it was determined that the protocols overestimate the median acceleration required to reach a damage state with a probability of 50%, with AC156 producing a more conservative estimation of the required median acceleration for tall, slender elements (B/H = 0.33) and FEMA461 producing a more conservative estimation of the required median acceleration for short, squat elements (B/H = 1.00 and B/H = 2.00).

It was also concluded that for short, squat elements, AC156 provides a more conservative estimation of the mean acceleration required to meet relative slide damage states, meaning that the percent difference of the mean acceleration for AC156 is smaller than that for FEMA461. Lastly, it was established that AC156 produces fragility curves with smaller standard deviations than those produced by FEMA461. As a result, the shape of fragility curves generated from real building histories is always more closely modeled by the fragility curves based on FEMA461 compatible time histories rather than those generated from AC156 compatible time histories.

Acknowledgements

This work was accomplished through the NEESreu program, and the author wishes to acknowledge Gilberto Mosqueda and Rodrigo Retamales for their mentoring and teaching during the program. Additionally, the author acknowledges NEESinc, NEES@Buffalo, and the SEESL staff for their support.

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