Adaptation of the PBEE Framework: A building block for community resilience models

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ABSTRACT

When evaluating community resilience, there exists a need to create a framework that evaluates the effects of natural hazards on the built environment. An inherent challenge in creating such a framework is the feasibility of developing accurate structural models on a regional scale. As a response, this report details the development of a software framework that allows low levels of structural information to be converted into accurate structural models and proceed through the Performance Based Earthquake Engineering (PBEE) framework to perform a seismic loss analysis. Using the city of San Francisco as a testbed, the framework will focus on developing regional models to mitigate the effects of earthquakes on the built environment. Defining features of this framework are its use of observational data from the California Strong Motion Instrumentation Program (CSMIP) to validate the accuracy of the structural models through a root mean square optimization and then perform a subsequent seismic loss analysis using the FEMA P-58 methodology.

INTRODUCTION

Historically, the seismic performance of buildings has been evaluated in terms of how individual buildings perform in the event of an earthquake. It makes sense that this would be the case, as structural engineers typically focus on the design of a structure in an isolated context and do not particularly consider how surrounding buildings may affect its performance. Mieler et al. [2013] describe the need for creating a framework that evaluates the effects of natural hazards on the built environment as a community as opposed to the performance of a single building without consideration of its interaction with surrounding structures. As is true in densely populated cities, a building does not exist in isolation and could be designed to a high standard such that it does not sustain any damage from an earthquake, however, if the building adjacent to it were to collapse onto it, then it becomes clear why building regulation needs to take the existing structural environment into consideration.

This report focuses on a potential approach of creating such a framework. One challenge is creating accurate structural models for an entire community without having all of the structural information necessary to complete the modeling process. The focus of this report is to detail the development of a framework based on the Pacific Earthquake Engineering Research (PEER) Center's Performance Based Earthquake Engineering (PBEE) formulation that converts low levels of structural information into accurate structural models to estimate seismic loss.

The current PBEE framework consists of four sequential analyses: hazard analysis, structural analysis, damage analysis, and loss analysis. Performance Based Earthquake Engineering essentially takes Engineering Demand Parameters (EDPs) such as peak roof acceleration and story drift ratio and translates them into decision variables such as repair costs, downtime, and other quantities that help building owners make decisions on how to invest in the design of their structure. Essentially, the framework allows for a more creative, safe, and economically advantageous design of structures beyond what the current building code requires for seismic design. In practice, this is typically a more design-oriented framework that engineers use for the design or retrofit of a structure.

The NHERI SimCenter's goal is to enable and improve the ability to simulate the effects of natural hazards on the built environment in order to increase the resilience of structures and communities against these natural hazards [2017]. However, the current PBEE framework evaluates the effects of earthquakes to the performance of a single building without consideration of how it may affect or be affected by surrounding structures. Thus, a community-oriented framework that allows for the quantification of a community's resilience to natural hazards as a whole is needed. The benefits of applying PBEE on a larger scale is that while on a smaller scale, the current PBEE framework promises increased seismic safety, reduced lifetime costs, and reductions in performance prediction uncertainty for single buildings [May 2006], if applied on a larger scale it will allow for the quantification of community resilience or risk. Expanding focus from a single building to an urban region requires a level of automation in the modeling process. The goal of the project is to develop a framework that allows low levels of information about a building to be converted into numerical models that yield accurate EDPs and seismic loss estimates.

For the purpose of this study, a 4-story steel, moment-frame building in the San Francisco area was studied as a baseline for outlining an optimization procedure that generates accurate structural models from low levels of structural information and observational datasets. These observational datasets consist of structural response data for past earthquakes and can be obtained through the California Strong Motion Instrumentation Program (CSMIP) online database through their Center for Engineering Strong Motion Data (CESMD). The optimization procedure was then adapted to other structures in seismically active regions with sufficient success of determining accurate parameters to replicate the existing structures' elastic properties. These linear-elastic models were then adjusted to include nonlinearity, allowing the framework to transition into structural analysis for several different hazard levels as well as the damage and loss assessment portion of the PBEE framework. Through this research project, the current PBEE framework was repurposed into a framework that both serves as a building block for community resilience models to be developed and is useful towards measuring community resilience on a larger scale.

MATERIALS & INSTRUMENTATION

No physical testing was done for this research, but observational datasets available through CSMIP formed the basis of this study and were used in the optimization procedure. CSMIP provides a large database of structures that have been instrumented with accelerometers on different floors and datasets of the structures' responses from past earthquakes that, in this framework, have been used to validate the accuracy of the structural models. Besides observational datasets, a variety of software was tied into the framework: MATLAB for data processing and optimization, the Open System for Earthquake Engineering Simulation (OpenSees) [McKenna et al. 2000] for structural modeling and analysis, and the Seismic Performance Prediction Program (SP3) for damage and loss analysis.

METHODOLOGY

The proposed adapted PBEE framework incorporates model optimization into the beginning of the current PBEE framework. The flow of this framework can be seen below in Figure 1.

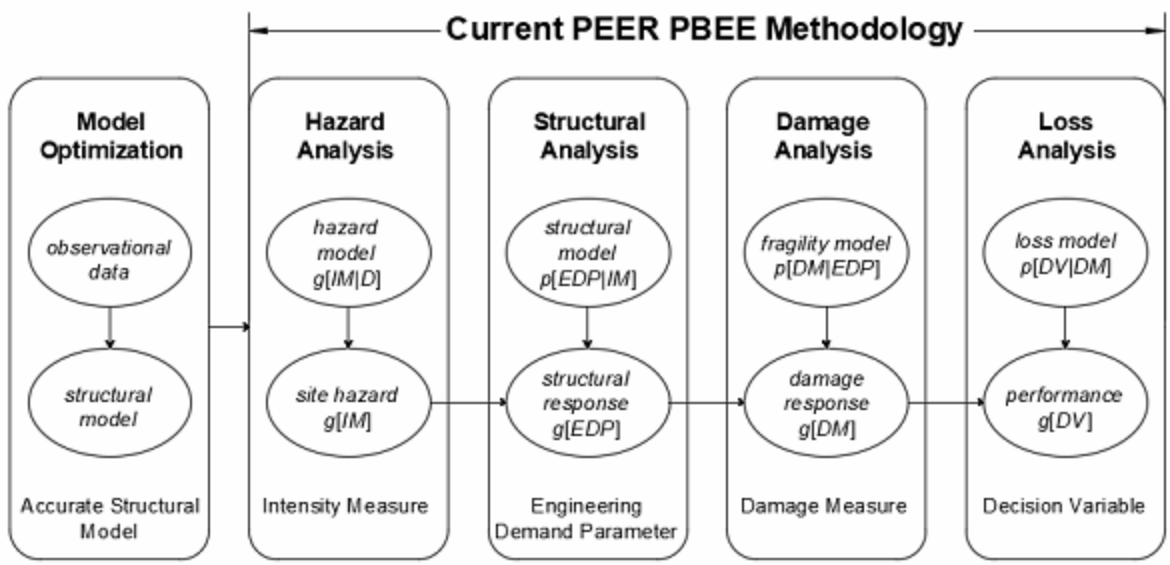

Figure 1: *Adapted PBEE Methodology [adapted from Porter 2003]*

The first step in repurposing the PBEE framework is model optimization, which utilizes observational data to develop accurate structural models from low levels of structural information. The next four steps follow the current PBEE framework: hazard analysis, structural analysis, damage analysis, and loss analysis. A rough outline of the flow of the framework can be seen in Figure 2.

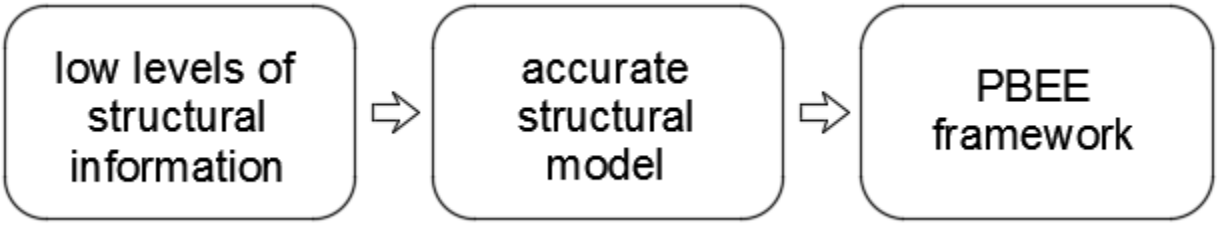

Figure 2: *Rough outline of the proposed framework*

The following assumptions were made in simplifying the structural models:

1. The models were multiple degree of freedom (MDOF) shear spring models as seen in Figure 3.
2. Models consisted of *n* masses and *n* springs where *n* is the number of stories.
3. Unit masses were used at each floor because they were not being optimized.
4. Vertical movement of the nodes was restricted because only the horizontal translations are optimized to the observational data.
5. The Giuffré-Menegotto-Pinto model with isotropic strain hardening was used to introduce nonlinear behavior in the model.
6. Rayleigh damping at the first two modes with 2% critical damping was used.

Figure 3: *Model simplification, actual building (left), simplified MDOF shear model (right)*

The setup of the framework is as follows:

1. Model Optimization: A pre-processing phase where the observational data is processed into a transfer function to identify the building period for the initial guess, the MDOF model is optimized to the observational data from CSMIP database to match the structure's elastic properties.
2. Hazard Analysis: A range of hazard levels are selected and scaled by considering magnitude, location, intensity, and input ground accelerations to be used for structural analysis [Porter 2003].
3. Structural Analysis: All hazard levels are run on the structural model adapted to behave nonlinearly, and the EDPs are recorded. The EDPs of interest are: story drift ratios, peak floor accelerations, and residual drifts, which are needed to move forward with damage and loss analysis.
4. Damage Analysis & Loss Analysis: The EDPs obtained from structural analysis are used in conjunction with component fragility functions and building inventories to determine damage measurements to facility components [Porter 2003]. The damage measurements are then transformed through loss analysis into "dollars, deaths, downtime, or other metrics" [Porter 2003].

The CSMIP database provides low levels of structural information that are used to set up the structural model as well as the damage and loss models, as can be seen Table 1.

Table 1: Low levels of structural information provided by CSMIP			
Latitude	37.6591 N	**Typical Floor Dimensions**	Same as base dimensions
Longitude	122.4388 W	**Design Date**	1972
Elevation (m)	18	**Instrumentation**	1976. 11 accelerometers, on 4 levels in the building.
No. of Stories above/below ground	4/0	**Vertical Load Carrying System**	Moment-resistant steel frame; 3-1/2" light-weight fill on metal decking (steel frames are distributed).
Plan Shape	Rectangular	**Lateral Force Resisting System**	Moment-resistant steel frame
Base Dimensions	144' x 197'	**Foundation Type**	Spread footings on piles (50'-70' deep); 8" RC slab on grade

Jirsa [2002] addresses the complexity in making PBEE feasible: the need for a lot of field data in order to calibrate and validate structural analysis programs. Model optimization thus incorporates the use of field or observational data and is designed in such a way that it relies on observational data from the CSMIP database to validate the MDOF models. The data from CSMIP is obtained at sensor locations located on certain floors as seen in Figure 4.

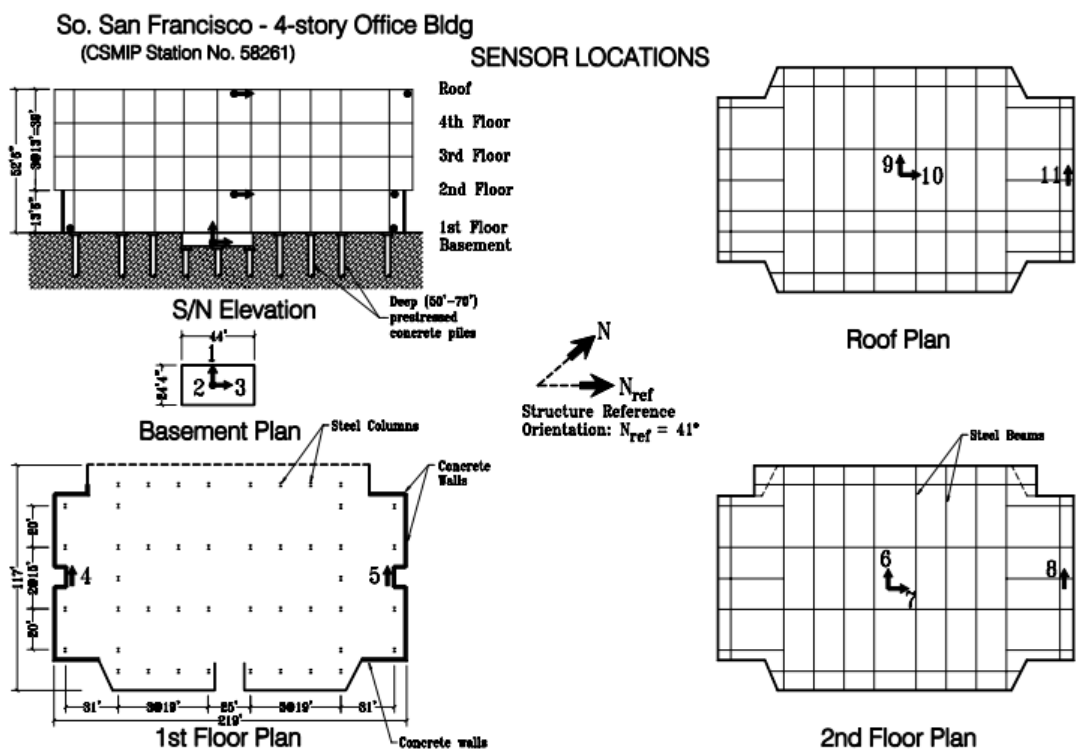

Figure 4: *Sensor locations provided by CSMIP*

Sensor locations may vary from building to building, but all buildings in the database consistently have accelerometers recording the base and roof accelerations in the north-south and east-west direction of the building. In this framework, the base acceleration and roof acceleration in a given

direction are extracted and processed to determine the building's fundamental period in both directions through a fast Fourier transformation (FFT) and transfer function. A MDOF shear model is then optimized or calibrated to match the elastic properties of the building using the observational datasets at the roof of the building. The observational data serves as a baseline for the framework, providing the accuracy and confidence in the structural model necessary to move forward with the PBEE methodology. For this study, the structural model was created in OpenSees. To optimize the model, the recorded base acceleration for a given earthquake was used as the input ground acceleration for analysis and the model's response was optimized to the observed roof accelerations for that earthquake. As additional validation, the optimized model was checked against other earthquake datasets recorded for the same building by comparing the model's roof acceleration responses to those observational recordings.

In the optimization procedure, linear time-history analyses were run on the model via OpenSees and the resulting root mean square error of the roof accelerations was optimized to a minimum. The optimization is set up such that the stiffness of each floor in each direction is randomized within a set range that is determined by the initial guess for the required stiffness of the model.

After the MDOF shear model has been optimized, hazard levels were selected and scaled according to building location. In its simplest form, hazard analysis for earthquakes consists of choosing hazard levels based on known, nearby faults. The location used in this study for hazard analysis can be seen in Figure 5. In this study, five hazard levels ranging from a 43-year return period to a 2475-year return period were chosen. Twenty ground motions per hazard level were chosen in order to fully characterize the hazard level, and examples of these ground motions can be found in Figure 6. The following probabilistic uniform hazard spectrums used in this study were provided by Jack Baker (personal correspondence) for the San Francisco scenario:

1. 50% in 30 years (43-year return period)
2. 20% in 50 years (224-year return period)
3. 10% in 50 years (475-year return period)
4. 5% in 50 years (975-year return period)
5. 2% in 50 years (2475-year return period)

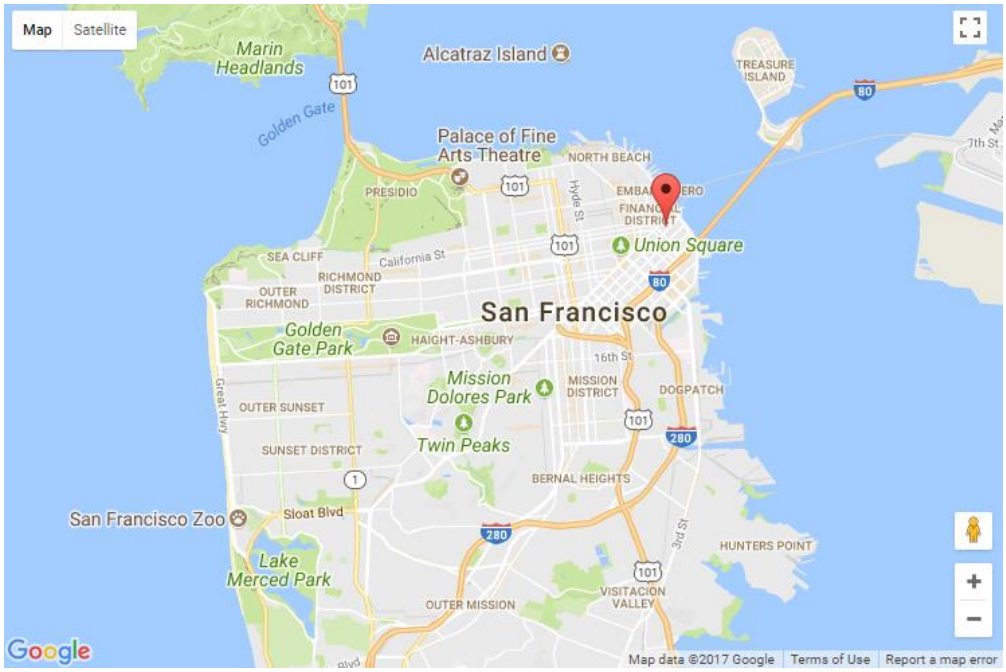

Figure 5: *Location used for hazard analysis [Google Maps]*

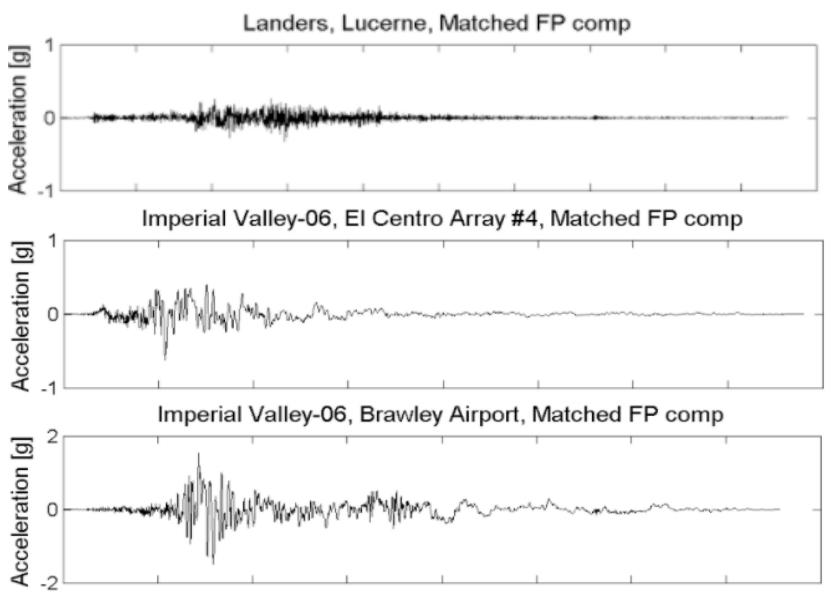

Figure 6: *Examples of selected hazard levels ranging from a 43-year return period to a 2475-year return period earthquake [prepared by Jack Baker 2014]*

After converting the linear elastic model into a nonlinear model, structural analysis was then performed using all of the input ground motion accelerations and the following EDPs were extracted: story drift ratios, peak floor accelerations, and residual drifts. OpenSees was used to conduct nonlinear time history analyses for each of the ground motions in order to obtain the EDPs to be inputted into SP3 for damage and loss analysis.

For damage and loss analysis, SP3 was chosen because it is suitable in making assumptions of a building's contents based on the type of building (commercial, residential, etc.) and from the low levels of structural information provided by CSMIP. A damage and loss analysis model was created in SP3 and user-defined structural analysis results were inputted and transformed into loss estimates in probabilistic terms such as percentage mean loss values for an earthquake with each of the hazard levels considered previously.

While this framework focuses on transforming individual buildings and their seismic response into loss estimations, it also serves as a tool to develop structural models from limited structural information and benefits the development of community resilience models.

RESULTS

The 4-story building was processed through the framework and the results for each step were documented below:

Model Optimization

The numerical model was optimized to the Morgan Hill (1984) earthquake – the first recorded earthquake for this building – and the initial and optimized story stiffness for the building under this earthquake are tabulated in Table 2. A visual comparison between the models using the initial guess story stiffness parameters versus the optimized story stiffness parameters can be seen in Figure 7 for both directions of the building.

Table 2: *Initial and optimized story stiffness from the Morgan Hill (1984) earthquake.*

Story	N-S		E-W	
	Initial (T=0.56 s)	**Optimized** (T=0.40 s)	**Initial** (T=0.55 s)	**Optimized** (T=0.39 s)
1	1.04e+03	2.00e+03	1.08e+03	2.01e+03
2	1.04e+03	2.05e+03	1.08e+03	2.12e+03
3	1.04e+03	2.24e+03	1.08e+03	2.40e+03
4	1.04e+03	2.38e+03	1.08e+03	2.04e+03

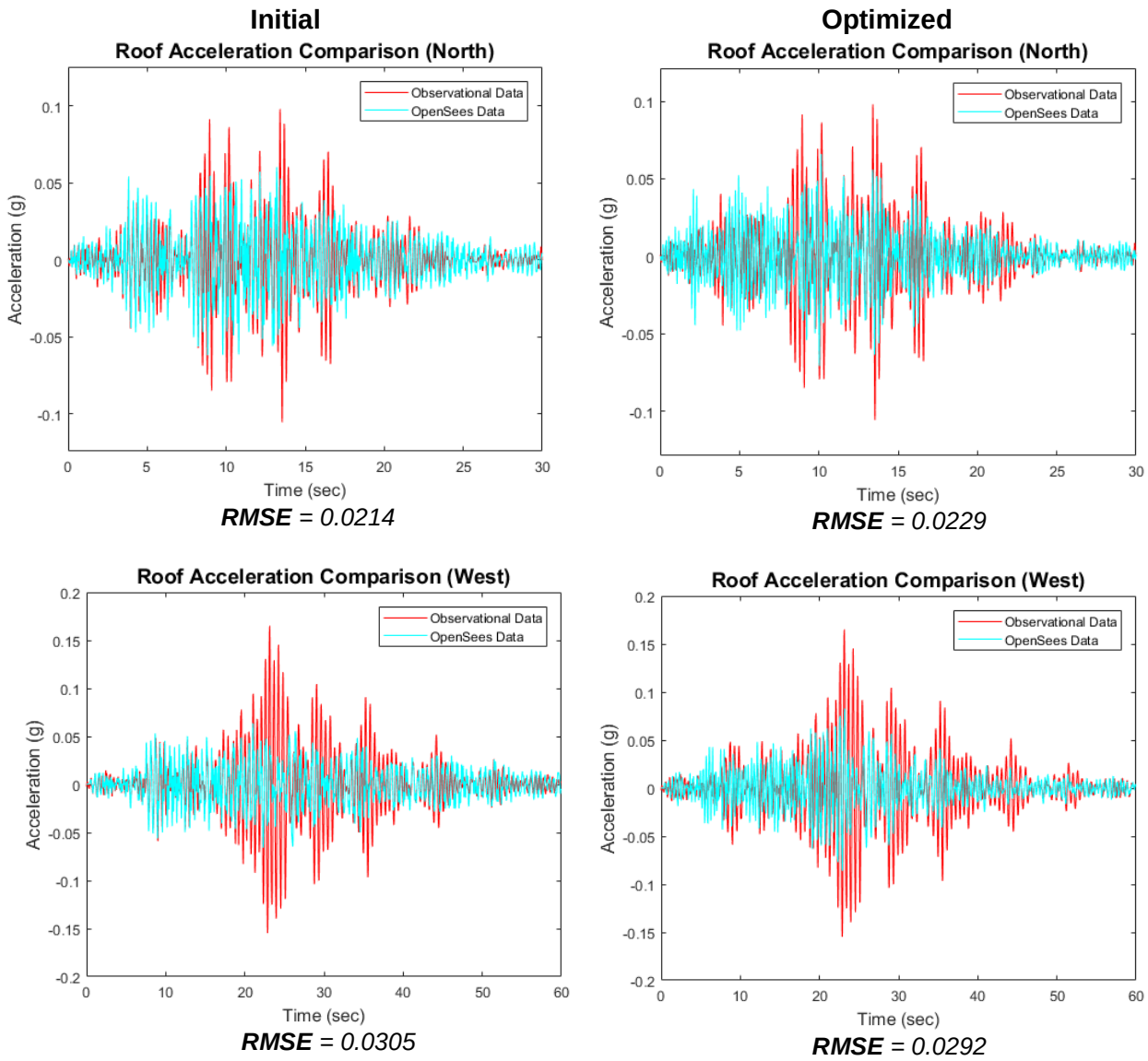

Figure 7: *Example of optimization results: initial guess (left) versus optimized parameters (right) comparison for the north-south (top) and east-west (bottom) response to the Morgan Hill 1984 earthquake.*

The optimized parameters from the Morgan Hill (1984) earthquake were then used for the model when running the Loma Prieta (1989) and South Napa (2014) earthquakes as additional validation in the accuracy of the model. Graphs comparing the observational data to the OpenSees model data to the two latter earthquakes can be found in Figure 8 while the root mean square errors for all three past earthquakes can be found in Table 3.

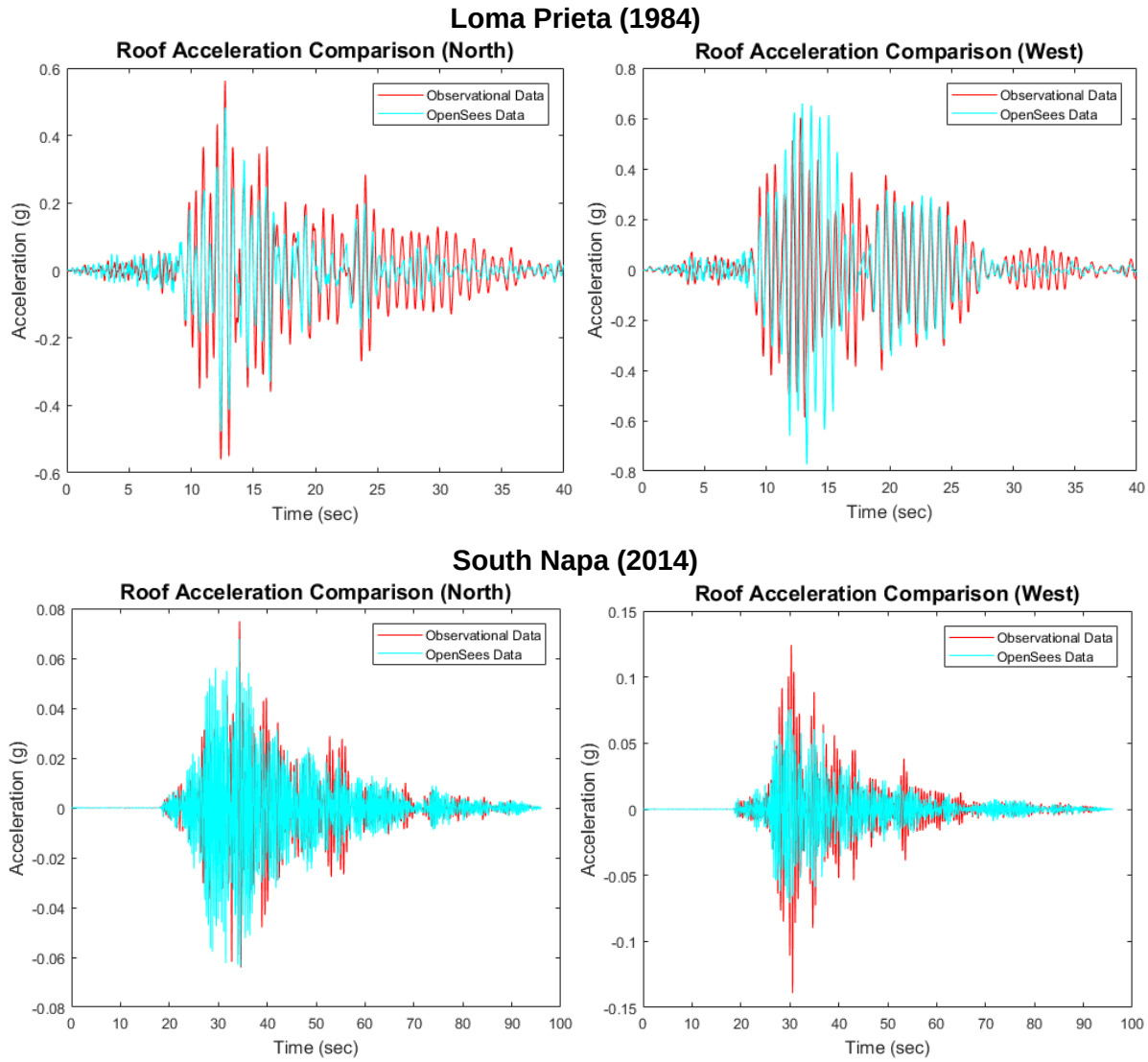

Figure 8: *Roof acceleration comparison between observational datasets and analytical model response using stiffness parameters optimized for the Morgan Hill 1984 earthquake for the Loma Prieta 1989 (top row) and South Napa 2014 (bottom row) earthquakes.*

Table 3: *Root mean square error results between observational data and optimized model for three past earthquake records using Morgan Hill (1984) optimized parameters.*

Earthquake Record	RMSE (N-S)	RMSE (E-W)
Morgan Hill (1984)	0.0229	0.0292
Loma Prieta (1989)	0.0674	0.1536
South Napa (2014)	0.0147	0.0177

Hazard Analysis

No results as all hazard levels were already prepared for this study – results can be found in Lai et al. [2015].

Structural Analysis

All five hazard levels (20 ground motions per hazard level) were run on the model via OpenSees. Each analysis included the two horizontal components of the ground motion. The peak floor accelerations for each hazard level, averaged among the 20 ground motions, can be seen in Figure 9. The maximum story drift ratios for each hazard level can be seen in Figure 10 where the maximum story drift ratio out of the entire story drift ratio time histories was selected. The residual drifts for the building were found by averaging the residual drifts for the 20 ground motions per each of the five hazard levels and can be visualized in Figure 11.

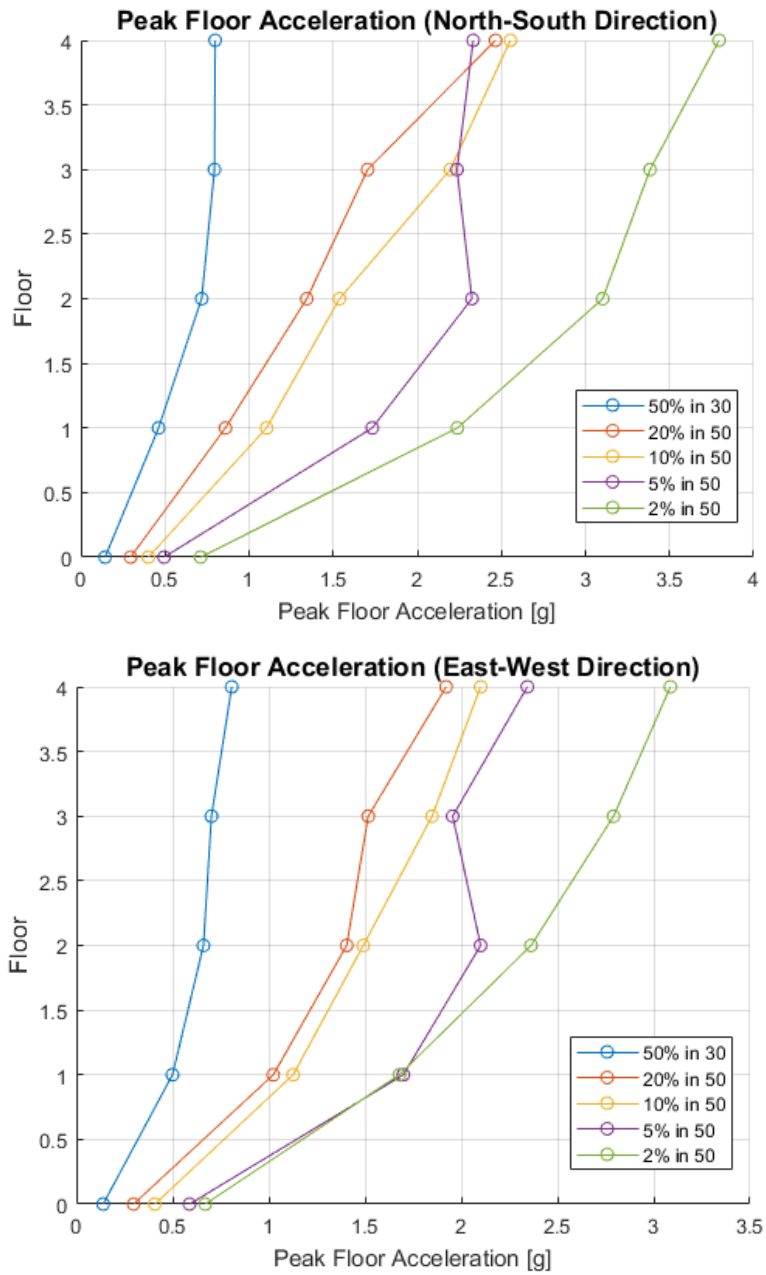

Figure 9: *Peak floor acceleration versus floor for the five hazard levels: N-S (top), E-W (bottom).*

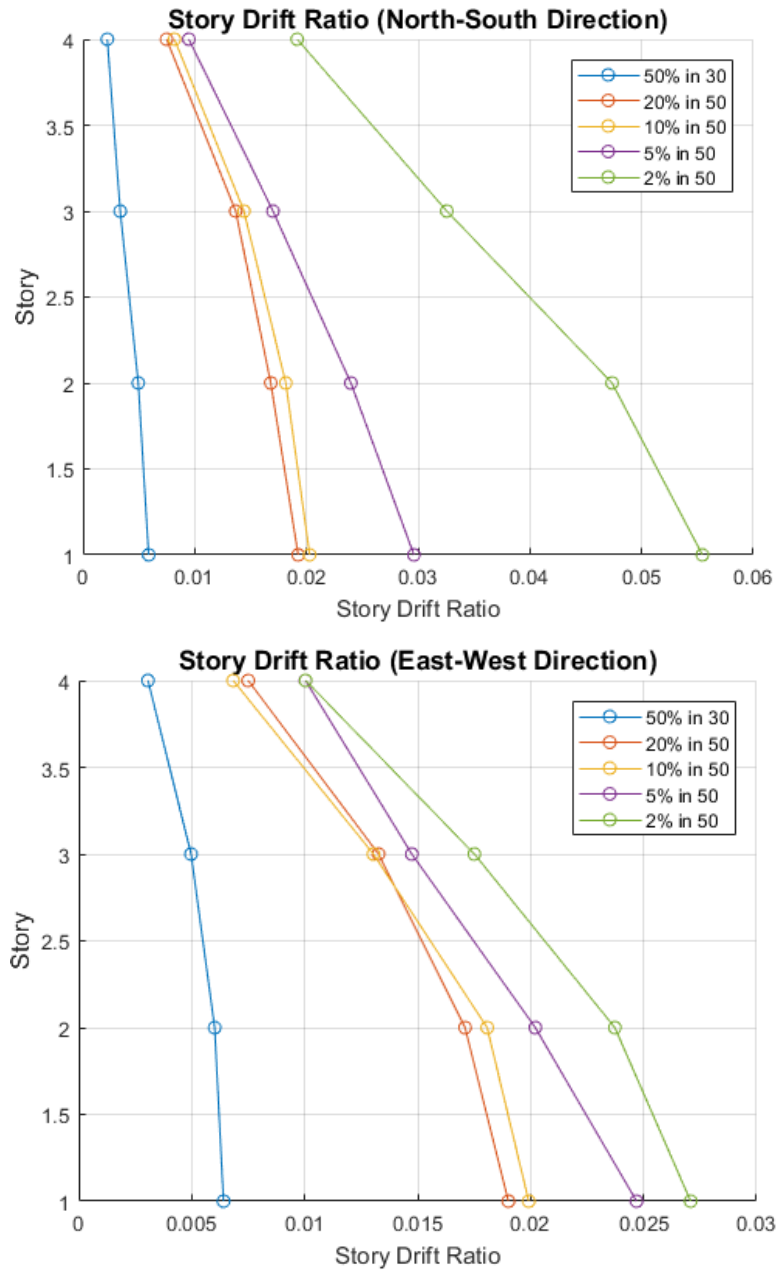

Figure 10: *Story drift ratio versus floor for the five hazard levels: N-S (top), E-W (bottom).*

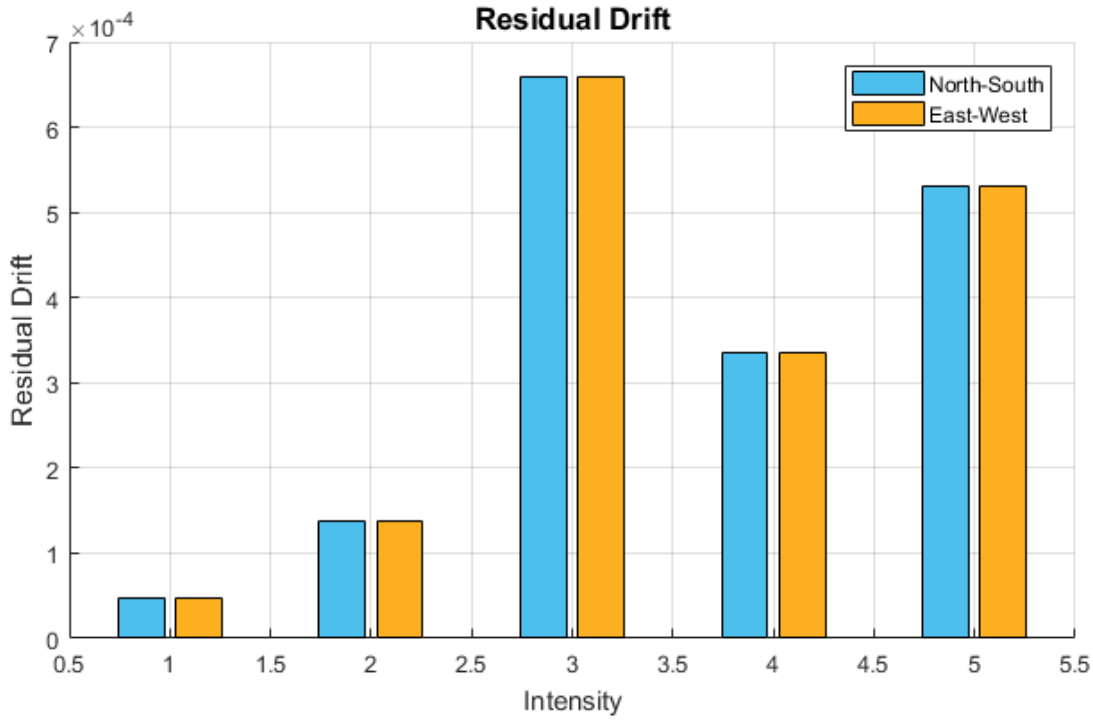

Figure 11: *Residual drift versus intensity level for the five hazard levels: N-S (blue), E-W (orange).*

Damage & Loss Analysis

Damage and loss analysis occurred in SP3 using the FEMA P-58 methodology and auto-populated structural and nonstructural components based on the square footage, structural system, height, and type of building. Extracted from SP3 are estimates on loss and repair time under each of the five different hazard levels which can be found below in Figure 12. For mean repair time, two estimations are provided using FEMA's P-58 parallel and series assumptions. The P-58 parallel estimation assumes there are enough workers to repair all of the floors simultaneously while the P-58 series estimation assumes repairs occur sequentially, floor by floor. Both of these are unrealistic scenarios, but serve as the lower and upper bounds for the approximate repair time of the building, respectively.

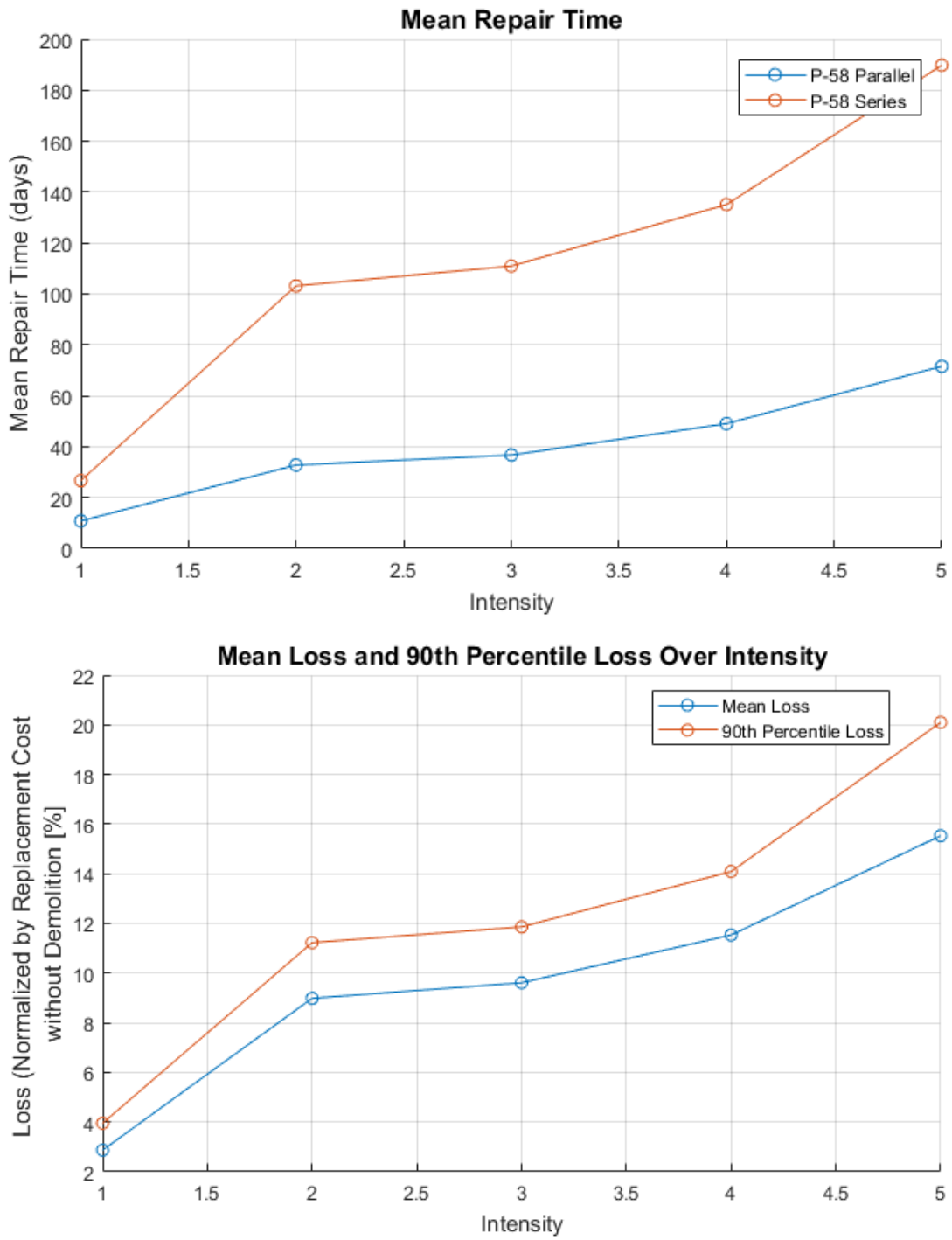

Figure 12: *Mean repair time (top) and loss (bottom) versus intensity of hazard level.*

Expected annual losses per building component can also be retrieved from SP3 as seen in Figure 13, which shows a breakdown of several different building components – both structural and nonstructural – and their percent contribution to the expected annual loss of the building under the inputted hazard levels.

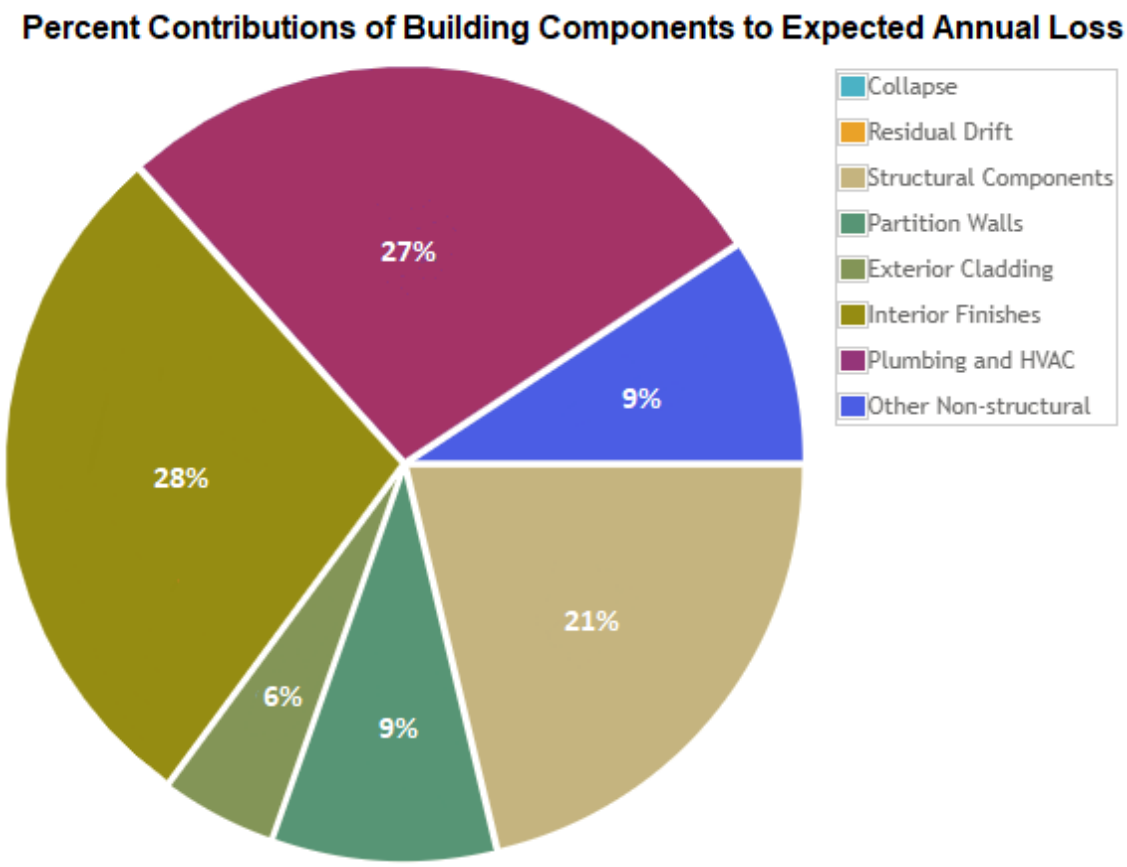

Figure 13: *Contributions of building components to Expected Annual Loss [graph from SP3].*

DISCUSSION

The goal of the model optimization procedure is to have the response time history match both the observational data in both phase and magnitude. Minimizing the root mean square error (RMSE) between the observational and analytical roof acceleration time histories was chosen as the objective function for matching the peak response. RMSE was chosen because larger errors are weighed more, thereby the objective function essentially puts more emphasis on matching the peak response values by minimizing RMSE. While the graphs in Figure 7 show that the optimized parameters for the Morgan Hill (1984) earthquake are not significantly better than the initial guess in terms of matching the peak responses, the optimized parameters help with matching the response phase. On the other hand, limitations in the optimization procedure's current setup where mass and damping parameters are not optimized most likely explain why the magnitude of response does not match for the Morgan Hill (1984) earthquake.

The optimization procedure is set up to only optimize the stiffness parameters of the model to the first earthquake recorded for the building and then was checked against the other earthquakes to determine whether or not the selected optimized parameters accurately and consistently represent the structural dynamics of the building. In Figure 8, it can be seen how the optimized parameters performed against the building's actual response to the Loma Prieta (1989) and South Napa (2014) earthquakes. Both the structural response and magnitude of the roof acceleration values match up fairly consistently. It should be noted that the responses are not expected to match up exactly

because of all of the model simplifications and assumptions. In this case, it can be said that the optimization procedure – which optimized to the linear elastic properties of the building – was successful and likely to yield more accurate demand parameters and loss estimates in the rest of the PBEE framework. Future refinement of the optimization procedure would be to expand it to optimize the damping constant for all of the modes of the building as well as account for more mode shapes of the building. The optimization procedure was set up on MATLAB but is extensible to other programs in order to enhance and expand the capabilities and efficiency of the optimization.

The structural analysis portion of the framework is set up to run all of the hazard levels (20 ground motions per hazard – 100 ground motions total) and return story drift ratios, peak floor accelerations, and residual drifts for all of the ground motions. These EDPs are output in the proper format from OpenSees to be directly uploaded onto SP3 and used for damage and loss analysis. SP3 has options for the user to choose hazard levels and run simplified structural analysis internally, however, the results do not take into consideration the accuracy of the model, thus the EDPs used to continue with damage and loss analysis are taken from this framework's structural analysis portion as the model has been optimized to observational data.

The benefits of using SP3 is that it does not require a lot of knowledge about the building's contents – both structural and nonstructural components. It requires only low levels of structural information, which benefits the ease of use, but may have large errors or uncertainty due to the assumptions made. SP3 auto-populates information on the components of the building using knowledge of the number of stories, floor heights, design date, and other information available in the CSMIP building metadata. Inputted into SP3 are story drift ratios, peak floor accelerations, and residual drifts from all of the hazard levels in two directions. SP3 outputs damage measurements as well as loss estimates such as downtime, repair cost, and annual loss. In Figure 13, it shows that structural components contribute a significant portion to the building's loss in comparison to the equipment in the building. Though building occupancy was specified in SP3, the specific equipment relating to the building occupancy is not generated in the model. Furthermore, given the reality that the building of interest is a hospital, it is expected that losses associated with the sensitive and expensive hospital equipment would have contributed more significantly to the annual loss of the building in comparison to the building's structural components. The estimates obtained from SP3 could therefore be improved if all of the building's contents and associated fragilities were known. Though there is no way to validate the accuracy of these estimates – as these hazard levels have not yet occurred for the building of interest, it serves as a reasonable prediction to how the building might perform in these events.

If this framework was used to create large amounts of structural models for buildings in a community or city, it could easily serve as a method to predict and quantify the performance of a city or its risk. On a regional scale, one method for obtaining these structural models is by using 3D GIS data, another form of low-level structural information database that offers geometric data of height and plan dimensions on all buildings [Zhu et al. 2006]. Zhu et al. [2006] produced a large database of MDOF structural models for the Bunkyo district of Tokyo using 3D GIS data and empirical equations in existing building codes to estimate the natural period of each building. Figure 14 shows a 3D view of the models produced by Zhu et al. [2006]. Though this project takes a different approach to validating the accuracy of the structural models, Figure 14 serves as a good

visualization of what is expected out of this framework: a large database of structural models to be used in accurately quantifying a community's resilience to earthquake hazards.

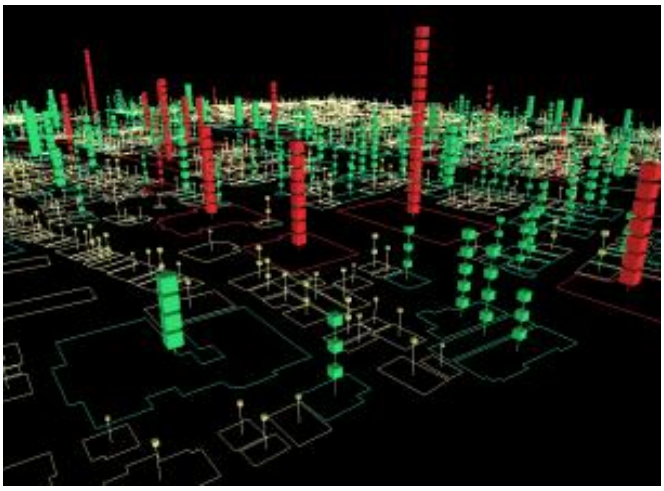

Figure 14: *3D view of structural models in Tokyo based on 3D GIS data [Zhu et al. 2006].*

The observational datasets provided by CSMIP are readily available and can be used to create accurate linear-elastic structural models and run seismic loss analyses on nonlinear adaptations of these models. This software framework seeks to provide a means to perform the former with a reduced amount of effort as it is structured to extract observational data from CSMIP, create the structural model, and optimize it to the extracted data with little to no human intervention.

This framework provides a method for taking low levels of structural information and creating accurate structural models through the optimization procedure, which validates the accuracy of the models by optimizing the models to observational datasets from past earthquakes. The example in this report sets precedence for more structural models to readily be developed from low levels of input in hopes of creating city-scale structural models to which the PBEE methodology can be applied to a city-scale in an attempt to understand community resilience. As of now, however, the feasibility of this framework depends on data collected from field studies – databases like CSMIP. While the accuracy of this framework fundamentally depends on the observational data provided by CSMIP, future goals would be to shift away from its dependence on field studies by creating a large enough database for Artificial Intelligence (A.I.) to be introduced into the framework. A.I. efforts would be directed towards reducing the amount of manual effort in creating these models by training it on the sufficiently large database of optimized models and thereby allow for a more automated modeling process. In addition, future goals include developing a well-integrated framework between all programs and steps used to increase the efficiency of the framework for future work, ultimately serving as a building block to which community resilience models can be developed.

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