TABLE OF CONTENTS

LIST OF FIGURES .........................................................................................................................3

1. Introduction .............................................................................................................................6

2. SUNY Buffalo Moment Frame Test .....................................................................................11

3. One-Story System Analysis .......................................................................................................15

   3.1 Scope .......................................................................................................................... 15

   3.2. Ground Motions ......................................................................................................... 16

   3.3 Cantilever ................................................................................................................... 17

      3.3.1. Model Setup .................................................................................................... 17

      3.3.2. Analysis Results ........................................................................................ 19

   3.4 Moment Resisting Frame ............................................................................................. 22

      3.4.1. Model Setup ..................................................................................................... 22

      3.4.2. MRF Prototype Design ................................................................................ 23

      3.4.3. MRF Analysis Results ................................................................................... 25

   3.5 Eccentric Brace Frame ................................................................................................. 28

      3.5.1. Model Setup ..................................................................................................... 28

      3.5.2. Prototype Design ........................................................................................... 29
LIST OF FIGURES

Figure 1.1. Reserve system behavior: (a) Experimental results for the 5th Story of a 0.3 Scale, 6-story CBF tested by Uang and Bertero (1986), as reported by Whittaker et al. (1990); (b) Idealized system force-displacement response of a braced frame with reserve system, showing three key parameters that affect reserve system behavior. ..............................................................8

Figure 2.1. Schematic of 4-story moment frame prototype on which 1:8 scale test frame was based. .............................................................................................................................................12

Figure 2.2: Buffalo Frame IDA Comparison (Lignos 2008, Figure 7.26).................................12

Figure 2.3. Buffalo Frame IDA Comparison (Experiment (NEES), Ruaumoko, OpenSees V.2.2.0). OpenSees analysis time step differs from input time step.........................................................13

Figure 2.4. Buffalo Frame IDA Comparison (NEES, Ruaumoko, OpenSees V.2.2.0). OpenSees analysis time step is the same as the ground motion input time step.........................................................14

Figure 2.5. Buffalo Frame IDA Comparison (Experiment (NEES), Ruaumoko, OpenSees V.2.2.2.e). OpenSees analysis time step is the same as the ground motion input time step ..........14

Figure 3.1. Ground motions and acceleration response spectra used for this study. ....................16

Figure 3.2. Cantilever system ........................................................................................................18

Figure 3.3. Comparison of reserve system pushover curves between SDOF in Figure 4 and the first floor reserve system for ¼ of the 9-story building .................................................................18

Figure 3.4. Cantilever IDA comparisons for GM4, GM8 and GM12 (Ruaumoko, OpenSees V.2.2.2.e) .......................................................................................................................................19
Figure 3.5. Cantilever time history comparison GM8 (Ruaumoko, OpenSees V.2.2.2.e). ............20

Figure 3.5. (continued) Cantilever time history comparisons for GM8 (Ruaumoko, OpenSees
V.2.2.2.e)........................................................................................................................................21

Figure 3.6. Moment Resisting Frame system. ..............................................................................22

Figure 3.7. CBF fracture force.......................................................................................................23

Figure 3.8. MRF moment diagram ...............................................................................................23

Figure 3.9. MRF IDA comparisons for GM4, GM8 and GM12 (Ruaumoko, OpenSees
V.2.2.2.e).......................................................................................................................................25

Figure 3.10. MRF time history comparison GM4, GM8 and GM12 (Ruaumoko, OpenSees
V.2.2.2.e).......................................................................................................................................26

Figure 3.10(Continue). MRF time history comparison GM4, GM8 and GM12 (Ruaumoko,
OpenSees V.2.2.2.e). ......................................................................................................................27

Figure 3.11. Eccentrically Braced Frame system. ........................................................................28

Figure 3.12. EBF moment diagram................................................................................................29

Figure 3.13. EBF IDA comparisons for GM4, GM8 and GM12 (Ruaumoko, OpenSees
V.2.2.2.e).......................................................................................................................................32

Figure 3.14. EBF time history comparison GM8 (Ruaumoko, OpenSees V.2.2.2.e).................33

Figure 3.15. Concentrically braced frame system. ........................................................................35

Figure 3.16. SDOF IDA comparisons for GM4, GM8 and GM12 (Ruaumoko, OpenSees
V.2.2.2.e).......................................................................................................................................36
Figure 3.17. Selected analysis time step for Ruaumoko model for GM12..................................38

Figure 3.18. SDOF IDA Comparisons for GM4, GM8 and GM12 with refined use of time steps for Ruaumoko. ........................................................................................................................................39

Figure 3.19. CBF IDA Comparisons for GM4, GM8 and GM12 with refined use of time steps for Ruaumoko. Reserve system strengths and stiffnesses are doubled in comparison with the IDAs shown in Figure 3.18......................................................................................................................40

Figure 3.20. CBF IDA Comparisons for GM4, GM8 and GM12 with refined use of time steps for Ruaumoko. Reserve system strengths and stiffnesses are halved in comparison with the IDAs shown in Figure 3.18......................................................................................................................40

Figure 3.21. CBF time history comparison GM8 (Ruaumoko, OpenSees V.2.2.2.e). ..............41

Figure 3.22. Time history comparison for CBF with double strength and stiffness, GM8 (Ruaumoko, OpenSees V.2.2.2.e).........................................................................................................................42

Figure 4.1. 9-story building designed assuming R = 3 as reported in Hines et al. (2009). ...........44

Figure 4.2. 9-story IDA Comparisons for GM4, GM8 and GM12..............................................45

Figure 4.3. Structural resurrection on the IDA curve of a 3-story steel moment-resisting frame with fracturing connections.........................................................................................................................47
1. Introduction

Designers in the East attempting to develop lateral systems that address moderate seismicity both safely and cost-effectively often find themselves constrained by code requirements that do not provide the flexibility that is available in the West (Hines and Fahnestock 2010). Many concentrically braced frame (CBF) buildings are currently designed using a response modification coefficient, $R$, equal to 3, which allows seismic detailing to be ignored. This approach has not been proven to guarantee acceptable seismic performance. Using $R = 3$ can result in design forces in the building and its foundations that are higher than forces resulting from wind loads, thereby increasing cost without clearly achieving elevated performance. Furthermore, it is not uncommon practice in moderate seismic regions to take a conservative approach to CBF design by specifying larger member sizes and larger forces than required for an $R = 3$ approach. In such cases, where the braces are oversized and the rest of the lateral system has not been explicitly proportioned according to capacity design principles, this type of conservatism can result in systems that are less safe than systems with weaker braces. In view of these limitations, new design approaches and system configurations may provide designers with opportunities both to ensure better seismic performance and to reduce cost. The philosophy behind such systems should enable designers to judge where added cost will most benefit system performance.

Empirical evidence indicates that steel braced frames possess appreciable reserve
capacity – in the form of gravity framing and gusset plate connections. These partially-restrained connection elements form a “reserve” moment frame system that can prevent sidesway collapse even when the primary lateral force resisting system (LFRS) is significantly damaged due to brace fracture. When required, reserve capacity can be enhanced without significant expense. As summarized by Hines et al. (2009), collapse performance of CBF systems that possess limited ductility appears to be impacted less by a system’s strength than by its reserve capacity. It is therefore proposed to reconsider the design of braced frames in low and moderate seismic regions as moderate-ductility dual systems. Such dual system behavior can be viewed from two different perspectives:

1. A stiff primary braced frame with a moment frame reserve system to prevent collapse in the event of brace failure.

2. A flexible moment frame stiffened by a sacrificial braced frame designed to withstand wind loads and to provide service-level drift control.

Figure 1.1. Reserve system behavior: (a) Experimental results for the 5th Story of a 0.3 Scale, 6-story CBF tested by Uang and Bertero (1986), as reported by Whittaker et al. (1990); (b) Idealized system force-displacement response of a braced frame with reserve system, showing three key parameters that affect reserve system behavior. Illustrates the idea behind moderate-ductility dual systems. In contrast to high seismic design, where system ductility is achieved through large component ductility, e.g., plastic hinges, brace buckling or brace yielding, moderate-ductility dual systems achieve system
ductility through the coupling of a stiff system with a flexible system. The stiff CBF system is expected to perform in a brittle manner; however, if the flexible system possesses sufficient strength, stiffness and ductility to prevent collapse, then adequate system ductility has been achieved.

Figure 1.1. Reserve system behavior: (a) Experimental results for the 5th Story of a 0.3 Scale, 6-story CBF tested by Uang and Bertero (1986), as reported by Whittaker et al. (1990); (b) Idealized system force-displacement response of a braced frame with reserve system, showing three key parameters that affect reserve system behavior.

The strength, elastic stiffness and ductility of the reserve system are represented by the numbers 1, 2 and 3 in Figure 1.1(b). It is possible to imagine a successful reserve system that has very little ductility (3) of its own. In this instance, system ductility would be achieved without relying on any component ductility. Figure 1.1(a) shows the performance of the 5th story of a 6-story, 0.3 scale, CBF dual system tested at the University of California, Berkeley in the mid 1980s. To this authors’ knowledge, these are the only published test results demonstrating the experimental performance of reserve system capacity based on a large-scale shake table test. Figure 1.1 (a) differs significantly from Figure 1.1 (b) in that the reserve system is nearly as stiff, and significantly stronger than
the primary CBF. Since the testing at Berkeley was part of a larger joint research venture between the U.S. and Japan in the 1980s, this design resulted from design criteria assembled to satisfy both U.S. and Japanese building codes that were then current. Considering the strength, stiffness and ductility of reserve systems as shown in Figure 1.1 (b) as fundamental to collapse performance of low-ductility steel CBFs, it becomes clear that the significant stiffness and strength discontinuities in such systems present challenges for the proper analysis of these systems. Furthermore, considering that collapse is the performance level in question for such systems, the ability of such analyses to predict collapse becomes of paramount importance. The interconnected nature of this problem, which includes low-ductility system performance, appropriate ground motion suite selection and careful consideration probabilistic methods for collapse risk assessment has been discussed at length by Hines et al. (2009, 2010, 2011). Ongoing large-scale component testing by Stoakes and Fahnestock (2010, 2011) has demonstrated that reserve capacity may be designed into standard braced frame gusset plate connections for relatively little cost, and work is underway to assemble the resources necessary for shake table testing of low-ductility CBFs with reserve systems at large-scale or full-scale. This report discusses the results of analytical work conducted since the printing of Hines et al’s 2009 paper. Recognizing the problematic nature not only of assessing the non-linear dynamic behavior of these systems but also of making these assessments up to the point of system collapse, the authors have attempted to re-frame the question of reserve system behavior on a fundamental level. Models have been simplified in order to isolate the key strength and
stiffness discontinuities between the primary CBF and the reserve system. These simple models have been constructed in two different software packages, Ruaumoko (Carr 2004) and OpenSees (2007) in an attempt to distinguish issues related to physical collapse from issues related to numerical convergence.
2. SUNY Buffalo Moment Frame Test

In preparation for modeling low-ductility CBFs with reserve systems, models were created to assess the performance of the 4-story small scale moment frame that was recently tested to collapse at the NEES facility at the State University of New York (SUNY) in Buffalo (Lignos 2008). In this research, a 1:8 scale 4-story moment frame was tested up to collapse, and compared to modeling results based on Drain 2D software. A schematic diagram for this test, as reported by Lignos (2008) is shown in Figure 2.1. Figure 2.2 shows the comparison, made by Lignos (2008) of the experimental and analytical incremental dynamic analysis (IDA) results. Based on these results, Lignos et al. concluded that “prediction of collapse is feasible using relatively simple analytical models provided that component deterioration is adequately represented in the analytical model” (Lignos et al. 2010, Summary).
Rotational springs were used to model the hinge areas of beams and columns. The P-Delta load was applied to the leaning column which was modeled as an elastic beam column element and connected to the frame using truss elements. The P-Δ geometric transformation was used to include the large displacement in the model. In the Ruaumoko
model, a bilinear inelastic hysteresis rule was used to model nonlinear behavior of the beams, assuming ILOS=0 implying no strength degradation. This model also has an elastic beam column element with large area to model the leaning column which is connected to the frame by a rigid link.

Figure 2.3 shows the IDA comparison between the experimental Buffalo frame, OpenSees and Ruaumoko. In this figure, OpenSees shows significantly larger collapse capacity with respect to two other results. The erroneous OpenSees results could be explained by the fact that the analysis time step ($\theta_{ta} = 0.005s$) differed from the input ground motion time step ($\theta_{igm} = 0.01s$). Figure 2.4 shows the IDA curve for the OpenSees model with compatible time steps. This model experienced convergence problems at even small scale factors (SF=0.2). After discussion of this issue with Lignos, the same model was run in a different version of OpenSees (Version 2.2.2.e). The IDA comparison
between OpenSees V.2.2.2.e, Ruaumoko and experimental data is showed in Figure 2.5. It can be seen that the IDA curves have significantly better correlation.

Figure 2.4. Buffalo Frame IDA Comparison (NEES, Ruaumoko, OpenSees V.2.2.0). OpenSees analysis time step is the same as the ground motion input time step.

Figure 2.5. Buffalo Frame IDA Comparison (Experiment (NEES), Ruaumoko, OpenSees V.2.2.2.e). OpenSees analysis time step is the same as the ground motion input time step.
3. One-Story System Analysis

3.1 Scope

In an effort to establish benchmarks for the comparison of Ruaumoko and OpenSees, a series of 1-story models were created. These models are:

- Single Cantilever
- Moment Resisting Frame (MRF)
- Eccentrically Braced Frame (EBF)
- Low-Ductility Concentrically Braced Frame (CBF)

While this research is concerned primarily with the performance of Low-Ductility CBFs, the difficulties in modeling such frames up to collapse led to the decision to establish baseline models of more ductile systems. Results from these models (presented in Sections 3.3 through 3.5) demonstrate a high degree of consistency between Ruaumoko and OpenSees for dynamic analysis of materially and geometrically non-linear systems. When compared to results for the low-ductility CBF in Section 3.6, the consistency in these more traditional, ductile systems helps to emphasize the uniqueness of low-ductility CBFs with reserve systems in terms of collapse performance and modeling. The large discontinuities in stiffness and strength experienced by these systems pose modeling challenges that simply are not present in more traditional systems.
3.2. Ground Motions

Ground motions for this study are taken from the previous study by Hines et al (2009) and Hines et al (2011). A detailed description of the suite of ground motions referenced in the 2009 paper can be found in Hines et al. (2011). Since the present study is concerned more with comparisons between analytical methods than with probabilistic performance assessment, it features only three of the ground motions from the previous studies: Ground Motions (GM) 4, 8 and 12. The acceleration response spectra (ARS) curves for these three ground motions, amplified according to a typical Boston Site Class D soil profile (Hines et al. 2011, Sorabella 2006) are shown in Figure 3.1.

![Figure 3.1. Ground motions and acceleration response spectra used for this study.](image-url)
3.3 Cantilever

3.3.1. Model Setup

The first system is a simple cantilever which forms a plastic hinge at the base, shown in Figure 3.2. This cantilever represents the reserve system of other simple models which is equivalent to the moment frame of the first story of the nine story model (for ¼ of the building, see chapter 4). To find the equivalent section a push over analysis was done on the first story of the nine story model and non-linear force-displacement response of the cantilever including P-Δ effects was calibrated to approximate the pushover curve for the first story of the 9-story building reserve system. Instead of adding a second leaning column to assume the remainder of the building weight, the area of the leaning column was increased to maintain axial stresses similar to the first floor graving framing columns. The resulting section is approximately 10.2 in. deep and 46 in. wide. The column vertical load is approximately equal to ¼ of the nine story building. The column material is steel, with $E = 29,000$ ksi, $F_y = 46$ ksi, and a linear strain hardening modulus of $E_{sh} = 290$ ksi. Note that these numbers were chosen to approximate the force displacement behavior of the first story reserve system, not to model an actual steel column. For this reason, it is not necessary for the cantilevered column in Figure 3.2 to be considered realistic in its own right. Figure 3.3 compares the pushover curves for the reserve system in Figure 3.2 and the first story reserve system for ¼ of the 9-story building. For the assessment of the first story reserve system, the second story was also modeled.
with braces intact so as to allow the gravity columns to contribute to the reserve capacity in addition to the gravity beams. In Ruaumoko, the plastic hinge length was set at half of the column depth or 5.1 in. In OpenSees, the column has 4 fibers along the depth and 16 fibers along the width.

![Figure 3.2. Cantilever system](image)

Figure 3.2. Cantilever system

![Figure 3.3. Comparison of reserve system pushover curves between SDOF in Figure 4 and the first floor reserve system for 1/9 of the 9-story building](image)

Figure 3.3. Comparison of reserve system pushover curves between SDOF in Figure 4 and the first floor reserve system for 1/9 of the 9-story building
3.3.2. Analysis Results

Incremental Dynamic Analyses (IDAs) for this system are shown in Figure 3.4. Performance of this system in Ruaumoko and OpenSees are very similar under GM4, GM8 and GM12, with Ruaumoko predicting slightly higher maximum drifts than OpenSees. Figure 3.5 shows response history analysis (RHA) results in terms of drift as a function of time for both programs under GM8. In this figure, it becomes clear that in addition to predicting slightly higher maximum drifts, Ruaumoko also predicted higher residual drifts. This observation is consistent with observations of other systems.

Figure 3.4. Cantilever IDA comparisons for GM4, GM8 and GM12 (Ruaumoko, OpenSees V.2.2.2.e).
Figure 3.5. Cantilever time history comparison GM8 (Ruaomoko, OpenSees V.2.2.2.e).
Figure 3.5. (continued) Cantilever time history comparisons for GM8 (Ruaomoko, OpenSees V.2.2.2.e).
3.4 Moment Resisting Frame

3.4.1. Model Setup

The next simple model which is shown in Figure 3.6 is considered to be a moment frame connected to a cantilever as a reserve system. The reserve system is the same as the cantilever column of Section 3.3. As it will be discussed more in the section 3.6 the braces in the 9-story model are assumed to fracture at a force of 297 kips at their connections prior to buckling. The moment frame is designed to resistant a lateral load equivalent to this fracture force. The material and hinge properties in OpenSees and Ruaumoko are the same as the cantilever model. The P-Δ geometric transformation was used to include the large displacement in the model.

Figure 3.6. Moment Resisting Frame system.
3.4.2. MRF Prototype Design

Figure 3.7. CBF fracture force

Figure 3.8. MRF moment diagram
Beam Design:

\[ M = 21060 \text{ k-in} \quad z_b = \frac{M}{F_y} = 458 \text{ in}^3 \quad \Rightarrow \quad W24 \times 162 \]

Column Design:

\[ Z_c = 1.1 \times 1.1 \times 468 = 566 \text{ in}^3 \quad \Rightarrow \quad W14 \times 311 \text{ (Column)} \]

\[ M = 21060 \text{ k-in} \]

\[ P = 450 \text{ kips} \]

For W14 X 311:

\[ A = 91.4 \text{ in}^2 \quad I = 4330 \text{ in}^4 \quad r = 4.2 \text{ in} \]

\[ \frac{kl}{r} = \frac{18 \times 12}{4.2} = 51.4 \quad F_e = \frac{\pi^2 E}{(kl)^2} = 107 \text{ ksi} \]

\[ F_{cr} = \left(0.658 F_e\right) f_y = 38.58 \text{ ksi} \]

\[ P_n = 38.58 \times 91.4 = 3526 \text{ kips} \]

\[ \frac{1.2 \times 450}{0.9 \times 3526} + \frac{8 \times 21060}{9 \times 0.9 \times 603 \times 46(1 - \frac{1.2 \times 450}{107 \times 91.4})} = 0.963 \]
3.4.3. MRF Analysis Results

The IDA comparison for GM4, GM 8 and GM12 can be seen in Figure 3.9. The figure shows a very good match between OpenSees and Ruaumoko results for all three ground motions. The results are shown up to collapse. As it can be seen in the figure, for the GM 4 and GM 8 the instability happen suddenly after the scale factor of 10 and 8 respectively. It’s not clear whether this instability is the result of numerical instability (which can suffer from the quality of code, time step, etc) or if it shows physical collapse. For GM 12, the model is stable up to 8 percent drift ratios in both programs which seems is correspondence to physical collapse. Figure 3.10 shows the drift time history comparison between OpenSees and Ruaumoko for GM 8. As it can be seen in the figure, the results shows very good match, However, for a scale factor 6 and larger, OpenSees shows residual drift after 40th second of ground motion while Ruaumoko doesn’t.

Figure 3.9. MRF IDA comparisons for GM4, GM8 and GM12 (Ruaumoko, OpenSees V.2.2.2.e).

25
Figure 3.10. MRF time history comparison GM4, GM8 and GM12 (Ruaomko, OpenSees V.2.2.2.e).
Figure 3.10(Continue). MRF time history comparison GM4, GM8 and GM12 (Ruaomoko, OpenSees V.2.2.2.e).
3.5 Eccentric Brace Frame

3.5.1. Model Setup

The eccentric braced frame which is shown in the figure 3.11 was designed for the fracture force of braces in the 9 story model (the same design force as moment frame discussed in the section 3.4.1). This model is also has the cantilever of Section 3.3 as the reserve system. In OpenSees all the members are modeled using fiber elements. In Ruaumoko, beam, column and braces are modeled as nonlinear members which forms hinge with the length of half of member size. The shear link in the Ruaumoko consists of six-bilinear rotational spring on each end of the element. More detail about the spring properties can be found in the thesis of Carlo C. Jacob (2010).

Figure 3.11. Eccentrically Braced Frame system.
3.5.2. Prototype Design

Figure 3.12. EBF moment diagram
**Brace Design:**

\[ \lambda M_p = 1.1 \times 1.25 \times 172 \times 46 = 11132 \text{ k-in} \]

\[ \lambda V_t = \frac{11132 \times 2}{60} = 371 \text{ k} \]

\[ \lambda V_{back} = 371 \times \frac{3}{12} = 92.7 \text{ k} \]

\[ P_{brace} = (371 + 92.7) \times \frac{21.6}{18} = 556 \text{ k} \quad \text{use HSS 10 \times 10 \times 1/2} \]

**Column Design:**

\[ \lambda V_t = 371 \text{ k} \]

\[ P_D = 470 \text{ k} \]

\[ P = 1.2 \times 470 + 371 = 935 \text{ k} \quad \text{use W12 \times 106} \]

**Beam Design:**

\[ z = \frac{\lambda M_p}{0.9F_y} = \frac{11132}{0.9 \times 46} = 269 \text{ in}^3 \quad \text{use W21 \times 122} \]
3.5.3. EBF Analysis Results

Figure 3.13 shows the IDA comparison between OpenSees and Ruaumoko for the mentioned EBF model for GM4, GM8 and GM12. As it can be seen in the figure there is a very good match between these two software results. The results for each ground motions are shown up to the scale factor which is associated with collapse. In this model, the shear link rotation of about 8 percent is considered as the collapse level. The drift time history comparison between OpenSees and Ruaumoko can be seen in the Figure 3.14. This figure indicates that the drift time history response of two models match well together. The only considerable difference between results is the residual displacements. As it can be seen in the figure after 40th second OpenSees shows some residual displacement while Ruaumoko doesn’t. As it discussed before, this difference in free vibration zone exists in all other models. It’s interesting to note that sometimes OpenSees shows this residual displacement and sometimes Ruaumoko does. Currently the reason of this difference is not obvious for the authors. In EBF this difference also can be seen in the vertical displacement of shear link end nodes which cause some differences in the shear – rotation hysteresis loop of the shear link.
Figure 3.13. EBF IDA comparisons for GM4, GM8 and GM12 (Ruaumoko, OpenSees V.2.2.2.e).
Figure 3.14. EBF time history comparison GM8 (Ruaomoko, OpenSees V.2.2.2.e).
3.6 Low-Ductility CBF

3.6.1. Model Setup

Based on the IDA and displacement time history comparison between OpenSees and Ruaumoko for ductile systems (section 3.3 through 3.5), the nonlinear behavior of these systems can be modeled with high confidence. Figure 3.15 shows the simplified model of the concentrically braced frame created to facilitate comparisons of non-ductile systems between OpenSees and Ruaumoko. The braced frame is similar to the braced frame on the first story of the 9-story building shown in Figure 4.1. Braces are assumed to fracture at a force of 297 kips at their connections prior to buckling. Both braces are modeled to fracture at the same time. This violates the idea that if the braces assume load from the floor above, the compression brace will fracture first, however it simplifies the behavior of the model and allows for more direct study of reserve capacity at a conceptual level. Brace fracture is modeled in Ruaumoko as described in Hines et al. (2009). Brace fracture is modeled in OpenSees by removing the brace from the model (death of the element) after it is subjected to the fracture force.
Figure 3.15. Concentrically braced frame system.

Tributary loads for the 9-story building are carried on the braced frame columns as masses, and the remainder of the building mass (for the ¼ building approximated here) is carried on a leaning column, whose non-linear force-displacement response including P-Δ effects was calibrated to approximate the pushover curve for the first story of the 9-story building reserve system.
3.6.2. Analysis Results

Figure 3.16 compares the IDA results using both programs. In general, the IDAs appeared to exhibit similar behavior, however, in many cases the Ruaumoko models would not converge at certain time steps. For this reason, the plots in Figure 3.16 show points only where convergence was achieved. This brought up two interesting considerations: (1) Ruaumoko had appeared to perform more accurately than OpenSees during the calibration study, whereas now OpenSees appeared to be converging more reliably; and (2) these IDAs raised the question as to whether it is possible to see collapse at a lower scale factor and then see resistance to collapse at a higher scale factor.

Figure 3.16. SDOF IDA comparisons for GM4, GM8 and GM12 (Ruaumoko, OpenSees V.2.2.2.e).

Figure 3.16, shows the IDA comparison for the time step of 0.001s. As it can be seen in the figure, Ruaumoko had convergence problem in some of scale factors. In an effort to try to improve convergence of the Ruaumoko models, time steps were varied across a spectrum: 0.001, 0.002, 0.003, 0.004, and 0.005. The result of this study was the
observation that under different ground motions, the Ruaumoko models converged better or worse under different time steps. Smaller time steps did not always yield more consistent results. This led to the intensive calculation of IDA curves assuming every one of the five time steps listed above. The IDAs plotted in Figures 3.18 through 3.20 reflect the selection of the most consistent IDA from the different time step runs.

Figure 3.17 shows the selected time steps for the GM12. To choose the appropriate time step, maximum displacements were compared for different time steps in each scale factor. For small scale factors (elastic range) the analysis was not sensitive to the time step variation. For example for the scale factor of 0.2 the maximum displacement was 0.002 for all time steps. In some of larger scale factors maximum displacements were identical or very close for different time steps. For example for the scale factor of 2 the maximum displacements were changed between 0.030 and 0.032. In this case the difference between maximum displacements is less than 7 percent which doesn’t have a significant effect on the IDA curve. In some other cases the results for one or two scale factors were very different with others. For example for the scale factor of 1, the model is unstable for the time steps 0.001, 0.002 and 0.004 and it is 0.0193 for time steps 0f 0.003 and 0.005. In these cases, the maximum displacement of the model in the working time step was used in the IDA curve. As it can be seen in figure 3.17 in some scale factors, the model was unstable for all the selected time steps.
Figure 3.17. Selected analysis time step for Ruaumoko model for GM12.

Figure 3.18 shows dramatic improvement in the convergence of the Ruaumoko models based on the selection of appropriate time steps. Currently, it is not clear why variation of the time step—sometimes as an increase, sometimes as a decrease, ensures better convergence of the Ruaumoko models. It is also not clear why the OpenSees models appear to be more stable in the context of this SDOF reserve system when they appeared to perform worse than the Ruaumoko models during the calibration study with the Buffalo frame. Figure 3.18 still shows some non-convergence at lower scale factors and then resumed convergence at higher scale factors. Currently, it also remains unknown whether this non-convergence is numerical or whether it represents physical collapse.
Figure 3.18. SDOF IDA Comparisons for GM4, GM8 and GM12 with refined use of time steps for Ruaumoko.

Figure 3.19 shows results very similar to those reported by Hines et al. (2009) when the reserve system strength and stiffness are doubled. Increasing the reserve system capacity yields dramatic improvement in collapse capacity assessed according to the method of incremental dynamic analysis. For each of the ground motions shown, doubling the reserve capacity yielded increases in collapse capacity of 50% to 100%. On the same note, cutting the reserve system strength and stiffness in half reduced collapse capacity uniformly by more than a factor of 2. Hence, while some mysteries regarding numerical convergence of these models with large stiffness and strength discontinuities remain unsolved, the relationship between reserve capacity and collapse resistance appears to be very clear, with consistent results based on independent models in different software packages.
Figure 3.19. CBF IDA Comparisons for GM4, GM8 and GM12 with refined use of time steps for Ruaumoko. Reserve system strengths and stiffnesses are doubled in comparison with the IDAs shown in Figure 3.18.

Figure 3.20. CBF IDA Comparisons for GM4, GM8 and GM12 with refined use of time steps for Ruaumoko. Reserve system strengths and stiffnesses are halved in comparison with the IDAs shown in Figure 3.18.

Figure 3.21 and Figure 3.22 shows the displacement time history comparison between OpenSees and Ruaumoko for the one story braced frame. As it can be seen in the figure, the results don’t match as well as ductile systems though the maximum displacements are close. It’s interesting to note that in this model Ruaumoko shows residual displacement but OpenSees does not.
Figure 3.21. CBF time history comparison GM8 (Ruaomoko, OpenSees V.2.2.2.e).
Figure 3.22. Time history comparison for CBF with double strength and stiffness, GM8 (Ruaomoko, OpenSees V.2.2.2.e).

Though the correlation shown in Figure 3.21 between two software packages is not as close as it was for the ductile systems, it may be possible to achieve better correlations with further refinements to the models, such as higher fidelity material modeling and updates to system stiffness at each time step. An integrated experimental and analytical work was done by Rodgers and Mahin (2004) to investigate the effects of various forms of degradation on the system behavior of moment frames and results indicated that
the details of response time history and residual displacements were very sensitive to the connection deterioration assumptions. Therefore, while the effects of sudden changes in the strength and stiffness of the system still remain for low ductility braced frames, better correlation may be achieved by using more similar material property in two software packages. Yazgan and Dazio (2006) showed that residual displacement also can significantly be influenced by element modeling approach. They modeled a reinforced concrete cantilever in OpenSees and Ruaomoko software packages using distributed and lumped plasticity elements and they concluded that residual displacement is sensitive to element properties. Also they showed that properly updating stiffness is crucial for estimating the residual displacement. Improvements to the low-ductility braced frame models based on these and other studies are part of ongoing work to establish appropriate modeling protocols for these systems.
4. Nine-Story System Analysis

4.1. Model Setup

The 9 story model is based on the SAC nine story building (SAC, 2000b), with two braced frame in each side (figure 4.1). The story height is 18 ft for the first story and 13 ft for other stories and the span lengths are 30 ft. The building is designed assuming R=3 for Boston, Massachusetts in accordance with IBC 2006 and ASCE 7-05 using Load Resistant Factor Design. The wind loads were determined using exposure B and the seismic load were determined using site class D and seismic design category B. More detail can be found Hines et al (2009).

Figure 4.1. 9-story building designed assuming R = 3 as reported in Hines et al. (2009).
Like the simple models, braces are assumed to fracture at the 297 kips force level. In Ruaumoko, beam and column are modeled using hinge element with the length of half of the member sections and in OpenSees fiber elements are used to model nonlinear elements.

4.2. Analysis Results

![Graphs showing IDA Comparisons for GM4, GM8 and GM12.](image)

Figure 4.2. 9-story IDA Comparisons for GM4, GM8 and GM12.

It is interesting to note, however, that in the case of the 9-story model, OpenSees appears to have had more trouble converging than Ruaumoko. This is similar to the observations made during the calibration study on the 4-story Buffalo frame, however it differs from the observations made on the SDOF studies, where OpenSees showed more consistent convergence. For this reason, it is difficult to conclude that one program should be preferred to another for this type of analysis, and it seems reasonable to insist that both programs continue to be used in future studies until a clear explanation can be offered regarding the reliable convergence of such models up to the point of collapse.
In general, the 9-story model exhibited lower collapse resistance than the SDOF model featured in Figure 3.15. There is, however, significant variation in collapse resistance between ground motions. The primary difference between the 9-story model and the SDOF model is the participation of higher mode effects. The nature of this participation, however, cannot be easily apprehended, because it does not correlate directly with general observations of the ARS curves shown for the three ground motions in Figure 3.1. For instance, Figure 3.1 shows GM12 to have significant high frequency content, whereas Figure 4.1 shows the 9-story structure with the highest collapse resistance under GM12. This observation is consistent with comments made previously regarding such systems (Hines et al. 2009, 2011) that their strength and stiffness discontinuities cause some level of chaotic behavior that is sensitive not only to the shape of the response spectrum, but also the sequencing of pulses and other ground motion signal characteristics that are not typically considered in suite selection.

It can be seen in all the IDAS for the 9 story and one story braced frame that 1) the curves show weaving behavior and 2) there is one or more collapse areas in some of them. The same behavior was reported by Vamvatsikos and Cornell (2002). They discussed four different types of IDAS which are 1) Softening Case, 2) a bit of hardening, 3) Severe hardening, 4) Weaving behavior. The 4th behavior is very similar to OCBF IDA curves. It’s interesting to be mentioned that they had all of these curves for a five-story braced frame and the responses ranging from a gradual degradation towards the collapse
to a rapid twisting behavior which shows different levels of strain hardening. Though a gradual behavior looks more intuitive, but the hardening in the IDA curves was seen before in different works. Chopra also reported this behavior for simple bilinear elastic-perfectly plastic systems. In the extreme case, the structure may collapse in one or more scale factor and then be stable in a larger one (Figure 4.2). This behavior can be seen in Figure 4.3 which is the same as the IDA curve of the 9 story model for the GM4 and GM12.

Figure 4.3. Structural resurrection on the IDA curve of a 3-story steel moment-resisting frame with fracturing connections
5. Summary and Conclusions

The primary objective of this research is to develop confidence in collapse performance prediction of low-ductility chevron braced frames as discussed by Hines et al. (2009), who modeled these systems using Ruaumoko-2D (Carr 2004). The 2009 exposed stiffness and strength discontinuities of these systems that raised the question of whether consistent results could be expected between different software packages. For this reason, the current validation study was engaged in order to compare Ruaumoko results with results from large scale shake table tests and results from OpenSees (2006).

The best approach to develop confidence in modeling of collapse performance would be to compare analytical results with large scale experimental results. In the absence of experimental data related to the collapse performance of low-ductility braced frames, this study considered experimental work on a 4-story 1:8 scale moment frame at the State University of New York (SUNY) in Buffalo (Lignos 2008). This moment frame was tested to collapse, and therefore provided a good opportunity to calibrate OpenSees and Ruaomoko models for prediction of side sway collapse under dynamic and P-D effects. Results showed that there is an acceptable correlation between Ruaomoko, OpenSees and experimental data.

The object of the next portion of this study (Chapter 3) was to demonstrate that different software packages exhibit a high degree of consistency when used to model
regular, ductile systems. Several simple models were considered: 1) Cantilever, 2) 1-story moment resistant frame (MRF), and 3) 1-story eccentric braced frame (EBF). The IDA and displacement time history comparison for all ductile systems showed a good match between OpenSees and Ruaomoko. These results coupled with the NEES Buffalo Frame Study indicated that: 1) it is possible to model the nonlinear behavior of ductile systems with high confidence and 2) there is a good match between maximum displacements for all models but the residual displacements can vary significantly. Note that Yazgan and Dazio (ETH Workshop, 2006) drew similar conclusions from comparative studies of reinforced concrete structural walls modeled with OpenSees and Ruaumoko, and indicated that it may be possible to address this issue.

Based on these promising results, a one story chevron braced frame with reserve system was modeled in OpenSees and Ruaomoko. There were three main differences between the results of this system compared with ductile systems which were:

1) The IDA curve of this system showed weaving behavior while ductile systems show softening or hardening behavior.

2) The correlation of IDA curves and time history displacement between OpenSees and Ruaomoko for this system was not as good as for ductile systems.

3) There were some convergence problems in the model obscuring whether non-convergence represented physical collapse or some other numerical instability.
These results indicated that differences between OpenSees and Ruaomoko could stem from the characteristic of brittle systems which is related to the brace fracture model and sudden change of stiffness and strength of the systems. For these reasons, it is important to exercise caution when modeling such systems, and it is advisable to report results from more than one modeling approach or software package. Ultimately, it will be very important to calibrate low-ductility braced frame models against experimental data.

In spite of the persisting uncertainties related to modeling these systems, it is still possible to observe significant correlation between reserve capacity and collapse resistance. Doubling the reserve capacity for the one story low-ductility braced frame in Chapter 3 increased the collapse capacity significantly. Conversely, cutting the reserve capacity in half reduced collapse capacity by more than a factor of 2.

Finally, the 9-story model was created in both software packages and the IDA curves were compared. This study only considered the 9-story R3 model so far. Results indicated that like one story braced frame model, the IDA curves showed weaving behavior and there were some convergence problems in the model.

Future work will include the following tasks:

- Further literature review of modeling issues, collapse behavior of building structures, and experimental performance of PR connections.

- Design a 9-story low ductility chevron braced frame with composite partially
restraint connections. This system will be designed with the assumption that moment frames will take all the earthquake loads and the braced frames are for drift control.

- Model this system in OpenSees and Ruaomoko and study the collapse behavior.

- Investigate the possibility of incorporating IDA results from all scale factors into reliability-based collapse assessment methods. Currently, only the scale factor associated with collapse is used for collapse performance assessment, and behavior at scale factors below the collapse level are ignored in the formal assessment procedure.

- Model braces such that they don’t fracture at the same time, and investigate other possible refinements to the system models, such as base plates, column splices, etc.
References:


Uang, C. M. and Bertero, V. V. (1986) Earthquake Simulation Tests and Associated Studies of a 0.3-Scale Model of a Six-Story Concentrically Braced Steel Structure, *Report No. UCB/EERC-86/10*, University of California, Berkeley, 363 pp.
