CHAPTER 5 ANALYTICAL MODEL

This chapter presents the development and calibration of a computer model for accurate performance assessment of BRKB-TMF system. A 2-dimensional model including gravity columns and P-Δ effect was used. Damping was defined as Rayleigh damping with 2% damping in the first and third modes. Material and component properties were defined mainly according to Seismic Rehabilitation of Existing Buildings (ASCE/SEI 41-06) with some adjustments. Figure 5.1 shows the analytical model used in this study. Components and system modelling are described in the following sections.

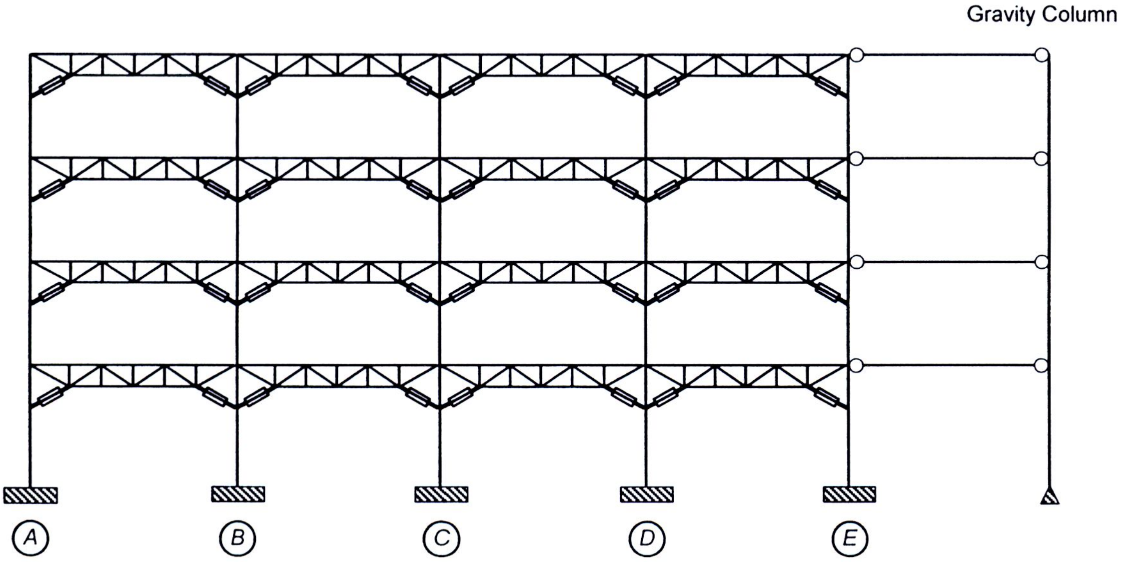


Figure 5.1 Analytical Model of an Archetype Structure

5.1 Component Modelling

Elements and components modelling are important aspects that affect system collapse evaluation. In this study, force-deformation characteristic of each element type, including strain-hardening, strength degradation and maximum deformation capacity were conservatively modelled according to ASCE 41 with an exception for those of BRBs and steel columns. The force-deformation characteristics for these two elements types were calibrated from experimental data and past research results.

All elements were modelled using actual strength to simulate the actual behaviour of the structure. The actual yield strength was calculated by multiplying the nominal yield strength, F_y, with a material overstrength factor, R_y. For all elements R_y was taken as 1.1, except BRBs which were modelled assuming that the yield strength was obtained from a coupon test. Therefore, R_y for BRBs is neglected.

5.1.1 Buckling Restrained Braces (BRBs)

BRBs were modelled as non-buckling elements with tri-linear load-deformation behavior. The stress-strain relationship was calibrated from a test result (Merritt et al., 2003b) having core length, elastic zone and stiff end zone of about 70%, 10% and 20% of total length, respectively. The configuration of BRB is illustrated in Figure 5.2.

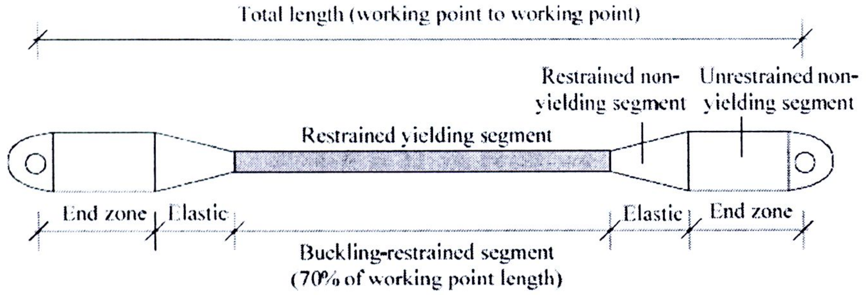


Figure 5.2 BRB Configuration

A load-deformation characteristic of BRBs was modelled as illustrated in Figure 5.3. In the first stage, deformation behavior was linear with an initial stiffness (K_0) up to the first yield deformation (D_y). Yield load (P_y) of in compression is assumed to be 1.2 times tensile yield load. Post-yield stiffness (K_H) was assumed to be $0.05K_0$ up to the deformation of $5D_y$. Maximum brace strain was assumed to be 2.8% when fracture occurred. With the assumed yield length of 70% of the total length, the maximum core strain was set as 4% (D_L). Fracture was modelled by a sudden strength drop when the core strain reached 4%. Minimal residual strength was assigned after fracture for numerical stability. The hysteretic loops from the test and the results from modelling are compared in Figure 5.4.

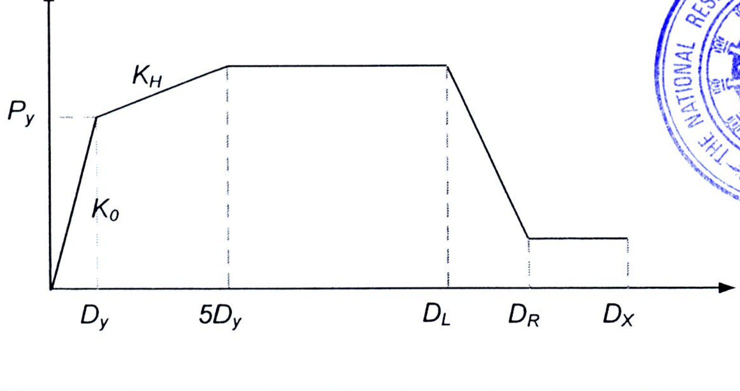


Figure 5.3 Load-Deformation Characteristic of BRBs

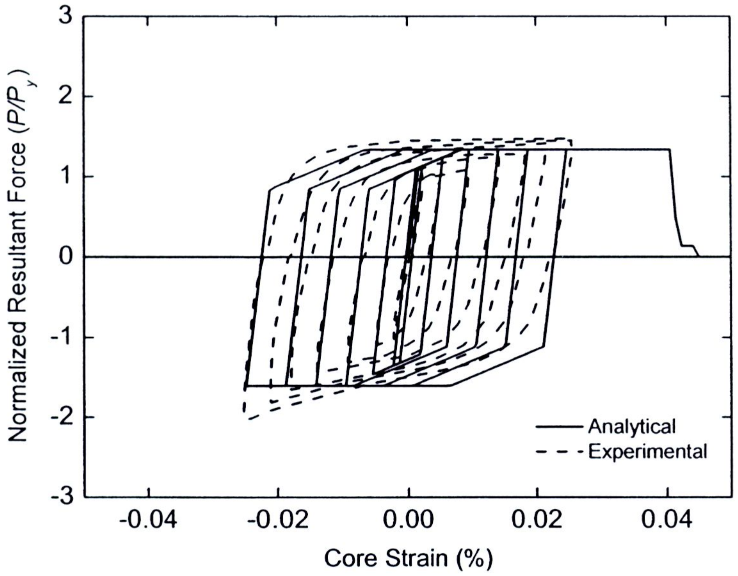


Figure 5.4 Comparison of Hysteretic Loops of BRBs from Computer Model and Test Results

5.1.2 Top and Bottom Chords

Chord members were modelled using beam elements with a lumped plastic hinge having a force-deformation relationship according to ASCE 41 as illustrated in Figure 5.5. The flange and the web compactness ratios of the designed chord sections in all floor levels were seismically compact; therefore, the maximum plastic rotation angle of a chord member was defined as nine times the yield rotation before strength degradation occurred. The residual strength was assumed to be 0.6 of the yield strength. The deformation limit was assumed to be eleven times the yield rotation.

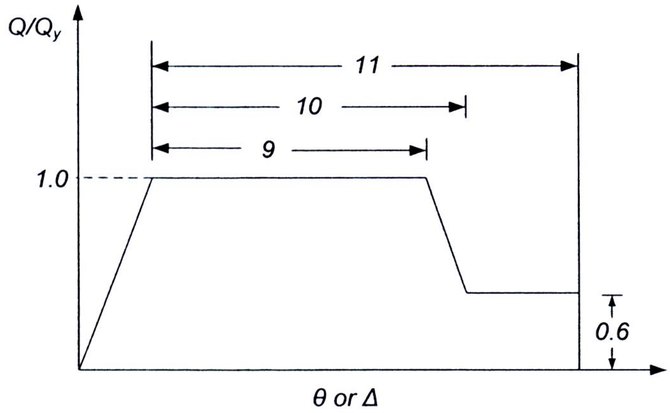


Figure 5.5 Force-Deformation Relationship for Top and Bottom Chords of the Trusses

5.1.3 Diagonal and Vertical Members

Buckling elements were used to model the diagonal and vertical members. The maximum strain values of both tensile and compressive deformation were assumed to be 10%. In tension, a load-deformation characteristic was assumed to be elastic-perfectly plastic. In compression, the response was elastic up to the buckling strength followed by gradual strength degradation. Strength dropped until the deformation reached 5 times the yield deformation and remained stable with a post-buckling strength of 30% of the buckling load. The load deformation characteristic for the buckling element is illustrated in Figure 5.6.

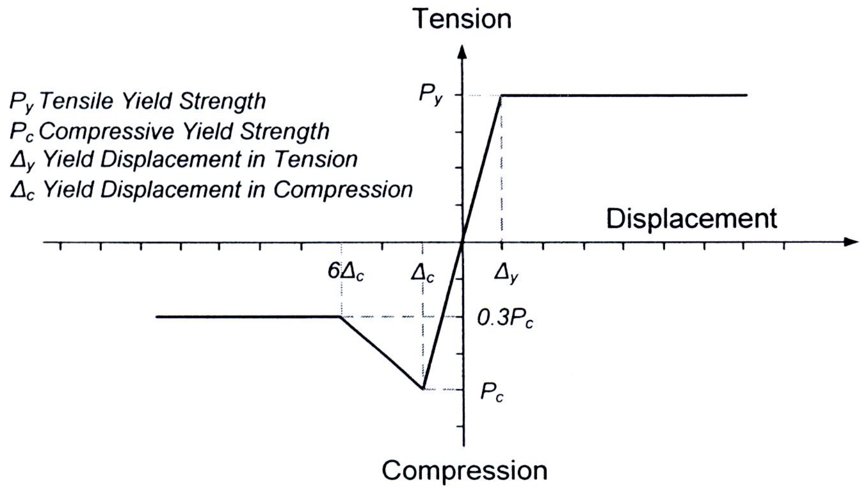


Figure 5.6 Buckling Element Model

5.1.4 Columns

Load-deformation characteristics for columns were assigned following ASCE41-06 with some adjustment. Recent research results (Newell and Uang, 2008) suggest that modelling parameters for columns in ASCE 41 are very conservative. The parameters corresponds to collapses at story drifts of only 3%-4% and no parameters are defined for columns subjected to axial force greater than 50% of column strength. Extra test results of steel wide-flange columns subjected to large drift and high axial load were presented by Newell and Uang (2008). The tests were done with columns having width-thickness ratios of 3 to 7 for flanges and 7 to 17 for webs. The test results showed that these columns could deform up to 10% drift before failure occurred. The normalized end moment versus strong drift ratio from one of the tests is illustrated in Figure 5.7. In the archetype structure, the width-thickness ratios of the designed columns ranged from 20 to 40 which were higher than those of the test specimens. Conservatively, a strength drop at 6% drift and fracture at 7% drift were used in all the analyses in this study.

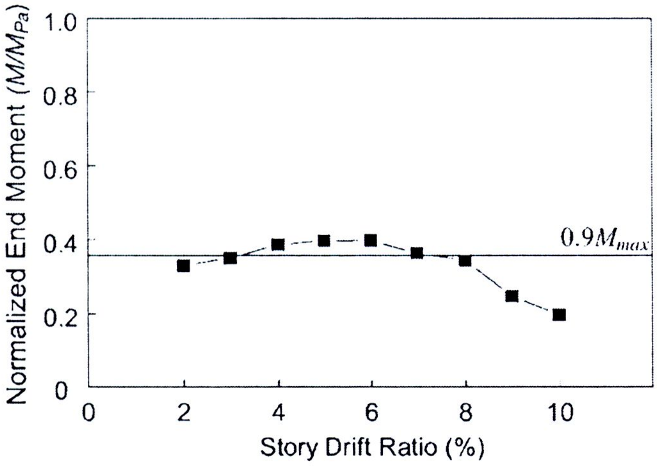


Figure 5.7 End Moment versus Drift Response Envelope (Newell and Uang, 2008)

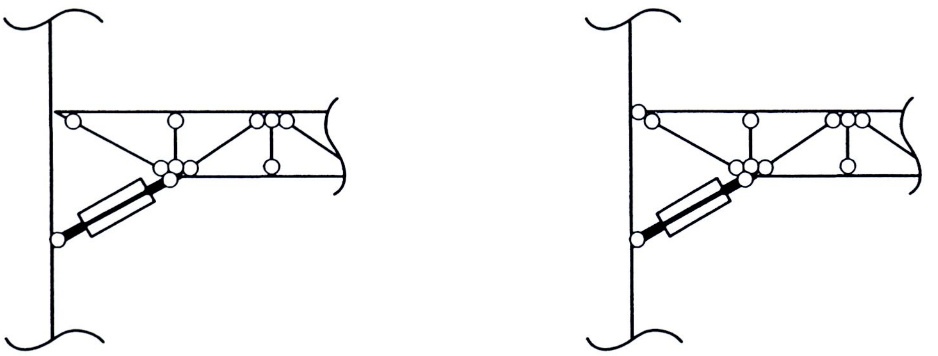
5.1.5 Non-Simulated Failure Modes

Critical phenomena that lead to collapse of BRKB-TMFs primarily include BRB fracture, leading to large story deformation, and large plastic rotation in the columns. These were explicitly modelled in the analyses. However, there were non-simulated failure modes which were not included into the model. To consider these non-simulated failure modes, post-calculation was used to detect them. In practice, many of these failure modes can be avoided by proper detailing and quality control during construction phase. These failure modes include:

- Fracture of gusset plates
- Column global buckling
- Local buckling of truss members
- Lateral torsional buckling and related failures

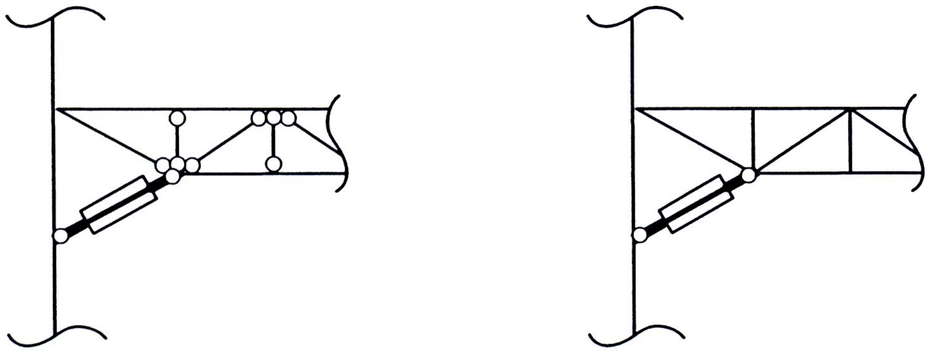
5.2 System Modelling

Generally, pin-ended elements are used for the analysis and design of a truss because they are simple and sufficiently accurate for gravity load cases. However, in seismic event, lateral forces may affect truss girders and may induce significant bending moment in the top chords. To ensure the accuracy of the model used in the analysis, the response of the structure for different modelling assumptions must be evaluated in order to find the most appropriate model. For this purpose, four different computer models (M1-M4) were created and assumed. Simple bar (pin-ended) and beam elements are used to model the chord, diagonal and vertical members. Detail of each model is illustrated in Figure 5.8 and can be summarized as shown in Table 5.1.



(a) Model 1 (M1)

(b) Model 2 (M2)



(c) Model 3 (M3)

(d) Model 4 (M4)

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Figure 5.8 Modelling of Truss Members

Table 5.1 Modeling of Truss Members

Model	Element Type		
	Chord	**Diagonal**	**Vertical**
M1	Beam	Bar	Bar
M2	Bar	Bar	Bar
M3	Beam	Bar*	Bar
M4	Beam	Beam	Beam

*all diagonal members are modelled as bar elements except first diagonal members are modelled as beam.

All 4 models were used in nonlinear static and nonlinear dynamic analyses. The results are as illustrated in Figures 5.9 and 5.10, respectively. As indicated, the base shear versus roof drift response of model M1 is very similar to that of the model M3. Furthermore, yield mechanism from these two models are also the same except the presence of some minor bending moments in the diagonal member at the exterior panel of model M3. Comparing between models M2, M3 and M4, the maximum base shear capacity of model M4 is the largest followed by that of models M3 and M2. Consequently, the roof drift response for model M4 is the smallest and the model M2 the largest, with the response from model M3 in between the two. In model M4, flexural yielding was detected in the first vertical members. However in all four models, the peak response values were very similar. The differences became significant only in terms of residual deformation. Since peak values are more important for collapse modelling, it was decided that model M3 should be sufficiently accurate as well as computationally efficient. Therefore model M3 was used in all subsequent analyses. It was also decided that a double angle should be used for the first vertical members to prevent out-of-plane bending since flexural moments were detected in model M4.

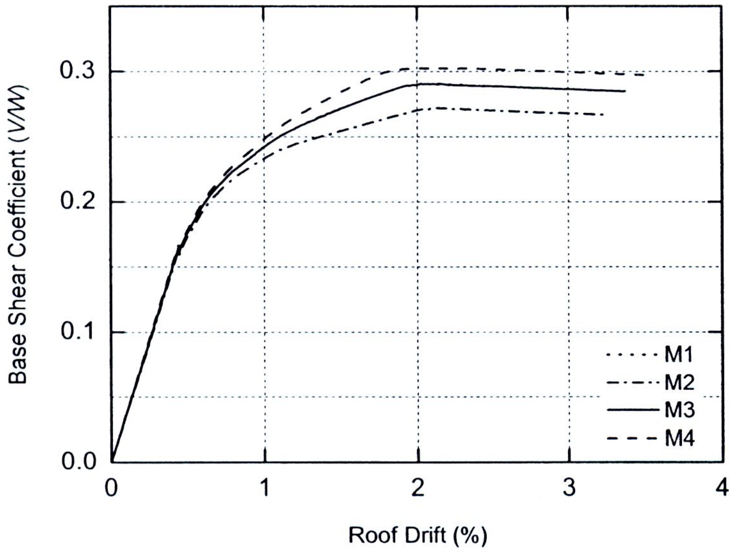

Figure 5.9 Base Shear-Roof Drift Plot Based on 4 Different Computer Models

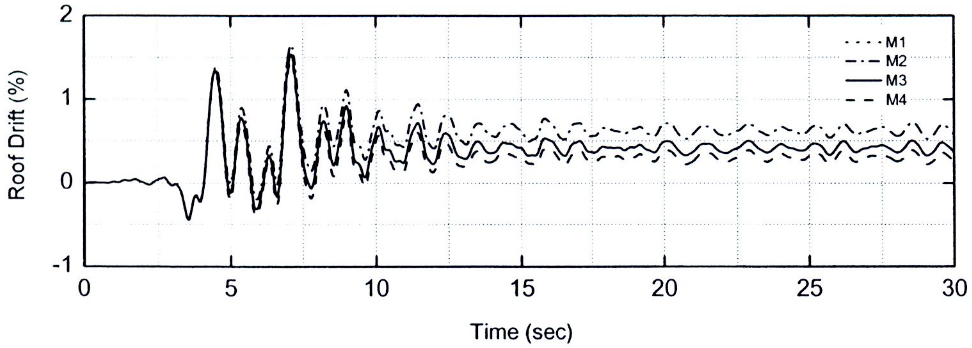

Figure 5.10 Story Drift Response of 4 Different Computer Models