CHAPTER 4 ARCHETYPE STRUCTURE

In this chapter PBPD design procedure of BRKB-TMF from Chapter 3 is used to design an example archetype structure (Goel and Chao, 2008) following the AISC Specification for Structural Steel Buildings (ANSI/AISC 360-2010) and AISC Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341-2010). Loads are computed based on ASCE/SEI 7-2010.

4.1 Building Configuration

The archetype structure is a typical 4-story steel building with perimeter BRKB-TMFs. Building dimensions are 36 by 54 meters with an overall height of 15.9 meters. The story height is 4.2 meters for the first story and 3.9 meters for the remaining stories. The building plan and elevation are shown in Figures 4.1 and 4.2, respectively.

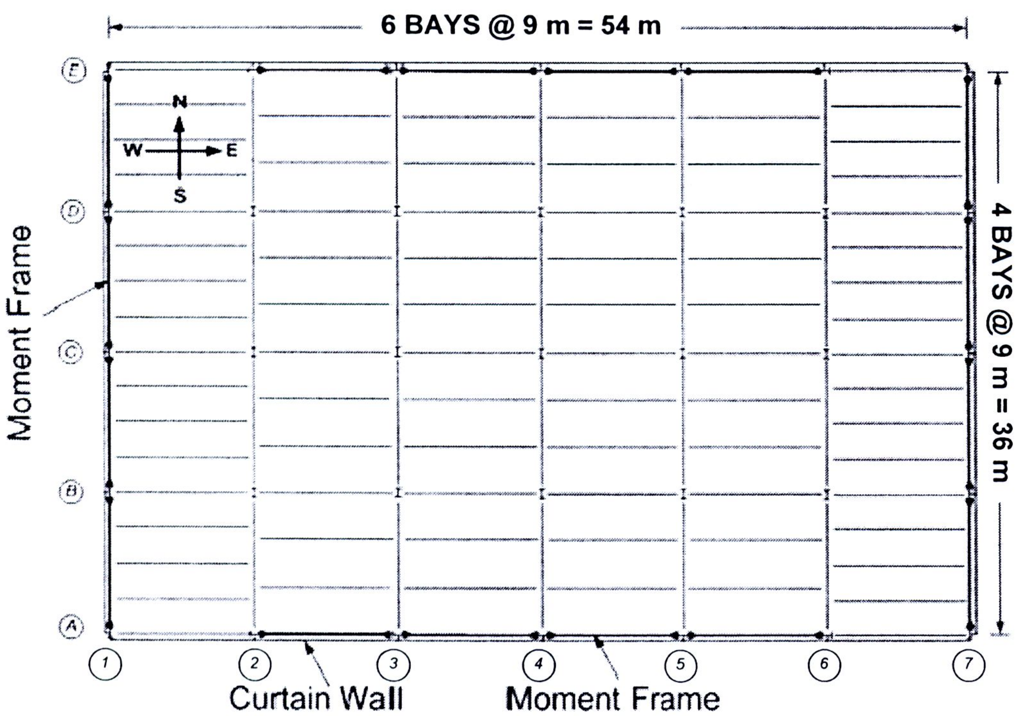

Figure 4.1 Building Plan (Goel and Chao, 2008)

The building configuration is the same as the example structure used by Goel and Chao (2008). This allows for performance comparison with other structural systems investigated earlier by Goel and Chao (2008).

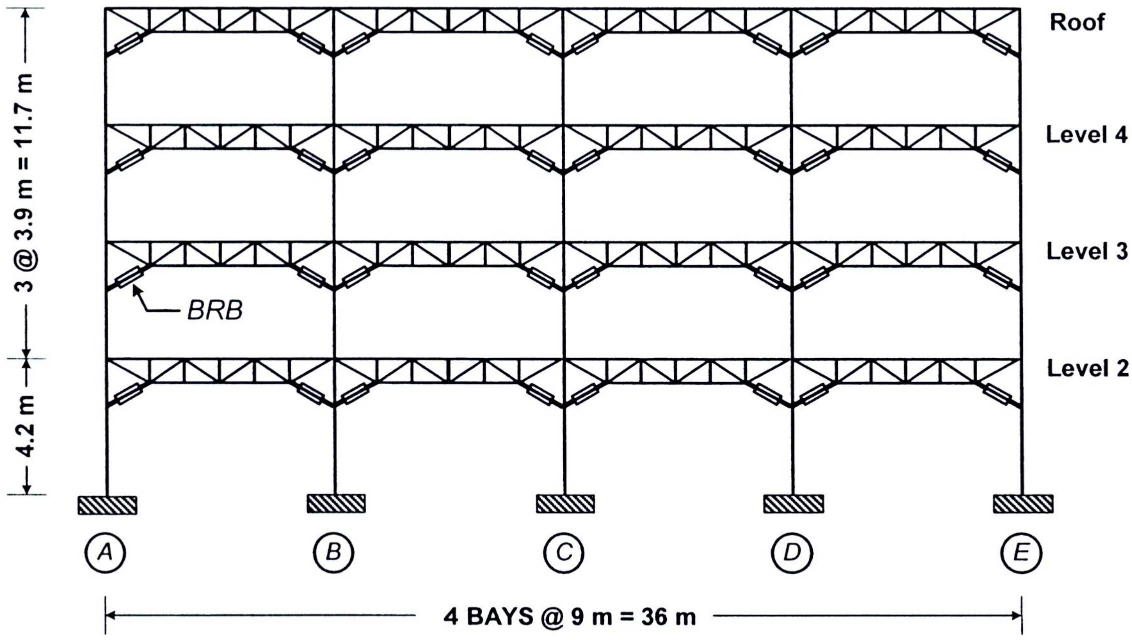

Figure 4.2 Building Elevation

4.2 Gravity Loading Criteria

Loads on trusses and columns are calculated based on ASCE 7-10 with live load reduction. The design gravity loads for roof and floor levels are listed in Tables 4.1 and 4.2, respectively. The loads follow those of the example frame used by Goel and Chao (2008).

Table 4.1 Design Roof Gravity Loads

Type	**Gravity Loads** (N/m^2)	**Seismic** (N/m^2)
Partitions	0	250
Allowance for Crickets	250	250
Roofing and Insulation	250	250
Allowance for Mechanical Equipment	250	250
2-1/2" LWC+W3 deck (5-1/2" total)	2520	2520
Deck deflection added fill allowance	200	200
Framing Self-Wt	Calculated	740
Fireproofing	150	150
MEP/Sprinklers	200	150
Clg/Lights	200	150
Misc.	200	150
Dead Load	4200	
Construction Load	2725	
Live Load (Reducible)	1000	
Construction	1000	
Mass Dead Load (Seismic)		5060

Table 4.2 Design Floor Gravity Loads

Type	Gravity Loads (N/m^2)	Seismic (N/m^2)
Partitions	1000	620
Floor Finish	50	50
2-1/2" LWC+W3 deck (5-1/2" total)	2520	2520
Deck deflection added fill allowance	200	200
Framing Self-Wt	Calculated	950*, 940**, 890***
Fireproofing	150	3
MEP/Sprinklers	200	3
Clg/Lights	200	3
Misc.	200	3
Dead Load	4460	
Construction Load	2725	
Live Load (Reducible)	2480	
Construction	1000	
Mass Dead Load (Seismic)		4790*, 4780**, 4730***

Note: *Level 2, **Level 3, ***Level 4

4.3 Design Base Shear and Lateral Force Distribution

The archetype structure is designed for two seismic hazard levels: the maximum considered earthquake (MCE) with 2% probability of exceedance in 50 years, and the design basis earthquake (DBE) defined to be 2/3 of MCE intensity. Ground motion intensity is calculated based on the design spectrum in ASCE 7-10. The target drifts were selected to be 3% for the MCE level and 2% for the 2/3MCE level. Yield drift ratio is assumed to be 0.75%. The maximum design base shear is 5894 kN from DBE ground motion. The base shear calculation is summarized in Table 4.3.

Table 4.3 Design Parameters for Example 4-Story BRKB-TMF

Design Parameter	2/3MCE	MCE
T (sec) Estimated	0.94	0.94
T (sec) Analysis	1.024	1.024
Sa (g)	0.64	0.96
Yield Drift Ratio θ_y	0.75%	0.75%
Target Drift Ratio θ_u	2.00%	3.00%
Inelastic Drift Ratio $\theta_p=\theta_u-\theta_y$	1.25%	2.25%
$\mu_s=\theta_u/\theta_y$	2.67	4.00
R_μ	2.67	4.00
γ	0.61	0.44
α	1.472	2.649
V/W	0.154	0.144
Design Base Shear (kN)	5894	5543

The design lateral forces are evaluated from the PBPD method. Table 4.4 shows the lateral forces for the study frame.

Table 4.4 Lateral Force Distribution

Floor	h_i (m)	w_i (kN)	β_i	F_i (kN)	F_i**/frame** (kN)	**Story Shear** (kN)
Roof	15.9	9791	1.000	2953	1477	1477
4	12.0	9467	1.516	1524	762	2239
3	8.1	9550	1.837	947	474	2712
2	4.2	9586	1.997	472	236	2948

4.4 Design of BRBs

The required strengths of BRBs were calculated from Equations 3-2 and 3-4. The results are summarized in Table 4.5.

Table 4.5 Required Strength of BRB

Floor	F'_i (kN)	h_i (m)	$F_i h_i$	β_i	$\beta_i N_{BRB}$ (kN)
Roof	369	15.9	5867	1.000	450
4	191	12.0	2292	1.516	680
3	120	8.1	972	1.837	827
2	58	4.2	244	1.997	899

Note: M_{pc} = 865 kN.m

Core strain limit of BRBs is an important criterion that must be carefully considered. Geometry of the trusses has a significant influence on brace deformation. The geometry of the truss must be chosen in accordance with the chosen target drifts. The BRB deformation at MCE target drift should be such that it is less than the deformation capacity of BRBs. From BRB test results, it can be observed that the deformation capacity of most BRBs in terms of maximum brace strain is in the order of 1.8-3.0%. By assuming a core length of $0.7L_{BRB}$, this results in the maximum core strain of about 3-4%.

To meet this core strain limitation, various geometries of the trusses can be investigated as shown in Table 4.6. The core strain demand can be calculated using Equation 3-1 presented in Chapter 3. Based on Table 4.6, the most suitable truss configuration can be selected such that the strain demand does not exceed core strain capacity of the BRBs. Figure 4.3 shows the selected dimensions of the truss moment frame for typical BRBs with core strain limit of 3-4%. Exterior panel length of 1.5 m and the depth of truss of 0.75 m were selected.

Table 4.6 Maximum Core Strain Demand for Various Truss Geometry

Exterior Panel Length (m)	**Depth of Truss** (m)		
	0.6	**0.75**	**0.9**
1.4	2.3%	2.7%	2.9%
1.5	2.2%	2.6%	2.8%
1.8	1.9%	2.3%	2.6%
2.1	1.6%	1.9%	2.2%

Note: θ_y = 0.0075; θ_p = 0.0225; core strain based on assume core length of $0.7L_{BRB}$

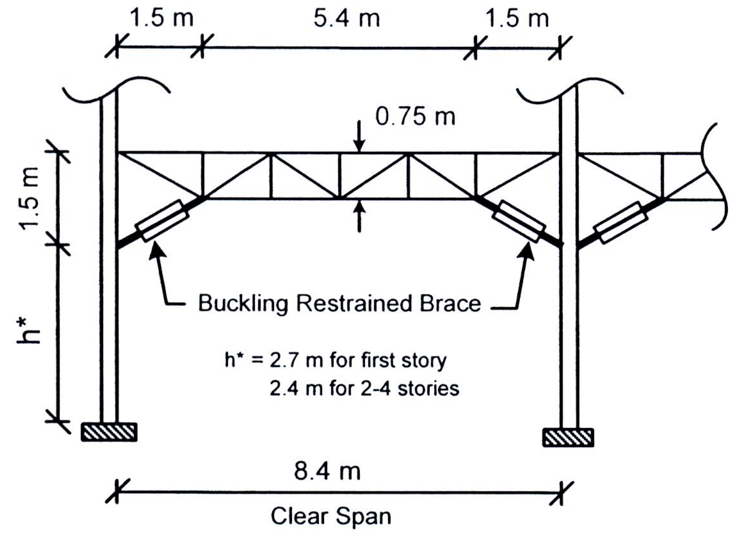

Figure 4.3 Selected Frame Dimension for Archetype Structure

4.5 Design of Truss Members

The trusses were designed to be elastic under gravity loads and adjusted BRB strengths. The adjusted BRB strengths are calculated by multiplying the BRB yielding strengths with overstrength factors (Equation 3-5)

$$P_{pr}^{+} = \omega R_y P_{ysc}$$

for tension and for compression (Equation 3-6)

$$P_{pr}^{-} = \omega \beta R_y P_{ysc}$$

In Equations 3-5 and 3-6, ω, β, and R_y are factors accounting for strain hardening, compression overstrength, and material overstrength, respectively. R_y has a value of 1 if the yield stress is determined based on coupon test.

The backbone envelope from a test is normally used to determine the tensile and compressive overstrength factors, ω and β. In this research, the envelope curve from Lopez and Sabelli (2004) was used as a model to determine β and ω factors. The envelope curve is shown in Figure 4.4. At 3% design target drift, the brace deformation was estimated to be 2.5% in terms of overall brace strain. Based on Figure 4.4, the overstrength factors of tensile and compressive forces are 1.5 and 1.75, respectively.

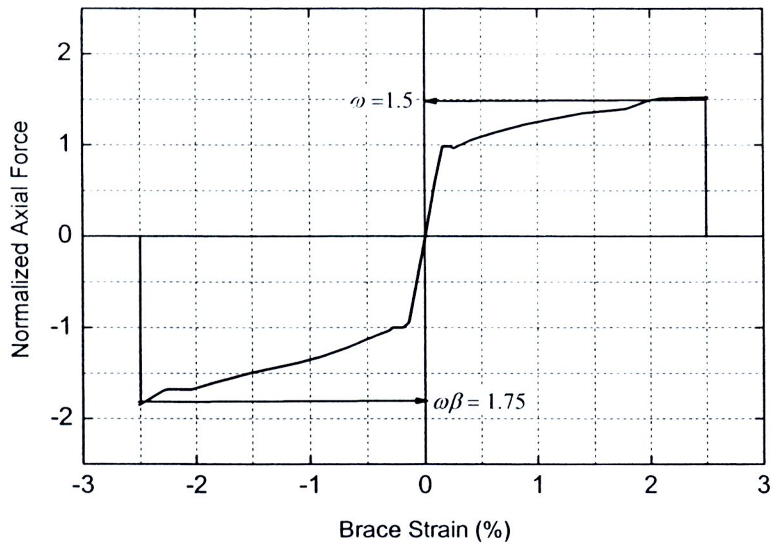

Figure 4.4 Hysteretic Backbone Envelope of Brace

Overall design of steel members was performed in accordance with AISC Specification and AISC Seismic Provisions. Chords and diagonal webs were chosen to be double channel sections connected at every one-third of the length. Nominal yield strength of 50 ksi (350Mpa) was assumed. The vertical members were angle sections except the exterior (outermost) vertical members which were double angle section. Nominal yield strength of 36 *ksi* (250MPa) was used for the vertical members. The reason why double angle section was used will be described later in Chapter 5. Tables 4.7 and 4.8 show the required strength and design results of a truss in each floor level. The design details are presented in Appendix C. The truss configuration is shown in Figure 4.5.

Table 4.7 Required Strength of Truss Members

Floor	**Chord** (kN)	**Diagonal** (kN)	**Vertical** (kN)
Roof	1241	721	125
4	1721	996	182
3	2020	1165	196
2	2171	1240	271

Table 4.8 Truss Section Design Results

Floor	**Chord**	**Diagonal**	**Vertical**	**Exterior Vertical**
Roof	2MC100x20.5	2C150x15.6	L89x89x7.9	2L89x89x7.9
4	2MC150x24.3	2MC150x17.9	L89x89x7.9	2L89x89x7.9
3	2MC180x28.4	2C7180x22	L89x89x7.9	2L89x89x7.9
2	2MC200x31.8	2MC150x22.5	L89x89x7.9	2L89x89x7.9

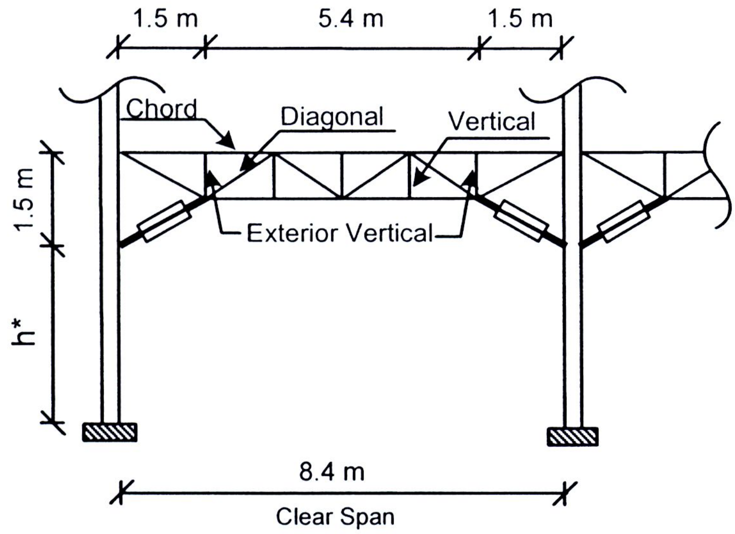

Figure 4.5 Section Configuration

4.6 Design of Columns

The columns were designed using the procedure presented in Chapter 3. From column tree analyses, re-balancing forces were calculated as shown in Table 4.9. Tension in braces produce uplift forces in exterior right column. Consequently, these forces were neglected for conservative design. P-Δ effect was included by using approximate second-order analysis according to AISC Specifications.

Table 4.9 Parameters for Column Design

Floor	α_i	Exterior Right		Interior		Exterior Left	
		$\alpha_i F_R$ (kN)	$(P_c)_i$ (kN)	$\alpha_i F$ (kN)	$(P_c)_i$ (kN)	$\alpha_i F_L$ (kN)	$(P_c)_i$ (kN)
Roof	0.501	531	92	880	46.8	348	92
4	0.259	274	111	454	61	180	111
3	0.161	170	111	282	61	112	111
2	0.080	85	114	140	63	56	114

Steel column design was performed every two floor levels. The design was done in accordance with AISC Specification and AISC Seismic Provisions. Columns with same depth size were selected to simplify fabrication process. The design results are shown in Table 4.10. More details can be found in Appendix C.

Table 4.10 Required Strength and Column Sections

Floor	Exterior Right			Interior			Exterior Left		
	M_u (kN.m)	P_u (kN)	Section	M_u (kN.m)	P_u (kN)	Section	M_u (kN.m)	P_u (kN)	Section
Roof	643	322	W610x174	1255	354	W610x285	613	504	W610x174
4	1193	381	W610x174	2457	457	W610x285	1264	747	W610x174
3	1306	405	W610x262	2704	541	W610x341	1398	914	W610x262
2	1306	420	W610x262	2704	585	W610x341	1398	1000	W610x262