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## **APPENDIX A**

Truss Configuration and Approximation of Strain Demand

This appendix describes the selection of the truss configuration and the BRB position in a BRKB-TMF. When struck by a severe ground motion, an inappropriate location may induce large deformation in BRBs, sometimes leading to fracture. Fracture of BRBs significantly reduces the base shear capacity leading to a large drift and instability in the end. It is shown in Chapter 6 that the BRBs are the key structural elements of this system. To ensure satisfactory behavior, the inelastic deformation expected to occur in the BRBs in a severe earthquake should not exceed the inelastic deformation capacity of the BRBs. This can be done by choosing an appropriate value for the target drift limiting the deformation demands of the BRBs.

### A.1 Selection of Truss Geometry

The configuration and the target story drift must be selected in relation with the strain capacity of BRBs. Inelastic deformation demand for a BRB can be calculated approximately from the target drift by assuming that the system deforms in a rigid-plastic manner after the mechanism has formed. By neglecting elastic deformation in the frame members, the plastic deformation of a BRB can be computed based on the truss configuration along with the law of cosines (Figure A.1).

Law of cosines:

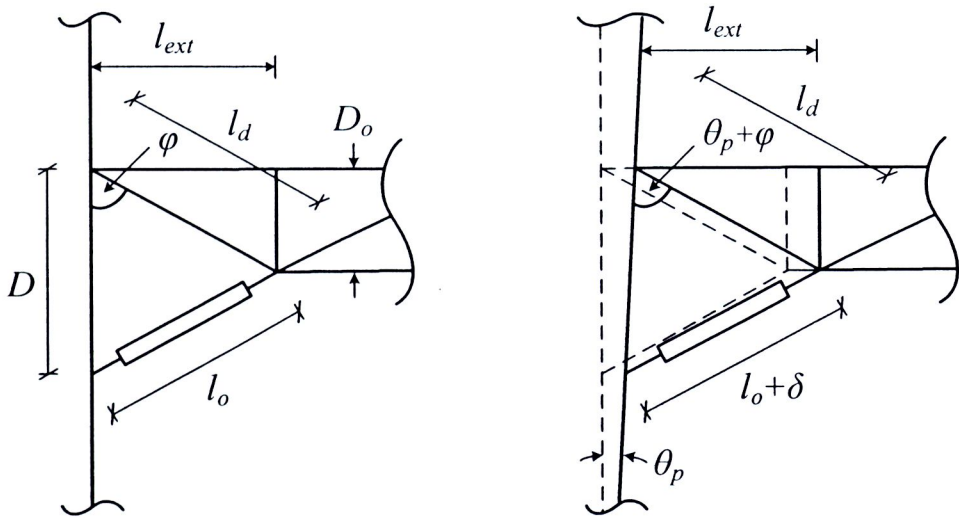
$$(l_o + \delta)^2 = D^2 + l_d^2 - 2Dl_d \cos(\theta_p + \varphi) \quad (\text{A-1})$$

where  $\theta_p$  is the plastic story drift angle,  $l_d$  is the length of the first diagonal member,  $\varphi$  is the angle between the first diagonal member and the column,  $D$  is the depth of the truss at the face of the column. The plastic strain is given by (Equation 3-1)

$$\varepsilon_p = \frac{\delta}{l_o} = \frac{\theta_p D \sin(\varphi)}{l_o}$$

The plastic core strain (assuming core length equal  $0.7l_o$ ) is

$$\varepsilon_{p,core} = \frac{\delta}{0.7l_o}$$



**Figure A.1** Application of Law of Cosine

As an example for the archetype structure described in Chapter 4, the target drift is selected to be 2% and 3% at DBE and MCE levels. The strain demands for various combinations of exterior panel of length and the truss depth can be calculated using the law of cosines for the governing MCE level. The calculation results are listed in Table A.1.

**Table A.1** Maximum Core Strain of Various Geometry at 3% Story Drift

Exterior Panel Length, $l_{ext}$ (ft)	Depth of Truss at Mid Span, $D_o$ (in)	Depth of Truss at the Face of Column, $D$ (in)		
		24	30	36
4.67	24	2.3%	2.4%	2.4%
	30	2.6%	2.7%	2.7%
	36	2.9%	3.0%	2.9%
5	24	2.2%	2.3%	2.4%
	30	2.5%	2.6%	2.6%
	36	2.8%	2.8%	2.8%
6	24	1.9%	2.1%	2.1%
	30	2.2%	2.3%	2.4%
	36	2.4%	2.5%	2.6%
7	24	1.6%	1.7%	1.8%
	30	1.8%	1.9%	2.0%
	36	2.0%	2.1%	2.2%

Note:  $\theta_y = 0.0075$ ;  $\theta_p = 0.0225$ ; core strain  $\approx \delta/0.7l_o$

Once the strain demands are calculated as shown, the best configuration can be selected based on the strain capacity of the BRBs. Proper selection of truss configuration does not rely only on the strain capacity of BRB, but the effective length of truss members must also be included in the consideration. Table A.1 indicates that using longer either  $l_{ext}$ ,  $D_o$  or the depth of truss at the face of a column requires lower strain capacity;

however, increasing the member length gives lower critical buckling strength. Furthermore, using a longer truss depth reduces the clear story height which impacts service function.

## A.2 Approximation of Strain Demand

Calculation of strain demand in section A.1 is accurate providing that elastic deformation is negligible. The proposed calculation method using the *Law of cosines* can be simplified. In this section, approximate calculation is presented. Assuming small deformation, the plastic elongation of the BRB can be computed from the deformed and undeformed shapes (Figure A.1).

$$(l_0 + \delta)^2 = D^2 + l_d^2 - 2Dl_d \cos(\theta_p + \varphi) \quad (\text{A-1})$$

$$l_0^2 = D^2 + l_d^2 - 2Dl_d \cos(\varphi) \quad (\text{A-2})$$

Subtract Equation A-1 from Equation A-2

$$\begin{aligned} (l_0 + \delta)^2 - l_0^2 &= -2Dl_d \cos(\theta_p + \varphi) + 2Dl_d \cos \varphi \\ (l_0 + \delta - l_0)(2l_0 + \delta) &= -2Dl_d \cos(\theta_p + \varphi) + 2Dl_d \cos \varphi \\ 2l_0\delta + \delta^2 &= -2Dl_d \cos \theta_p \cos \varphi + 2Dl_d \sin \theta_p \sin \varphi + 2Dl_d \cos \varphi \end{aligned} \quad (\text{A-3})$$

For small deformation,  $\delta^2 \approx 0$ ,  $\cos \theta_p \approx 1$ ,  $\sin \theta_p \approx \theta_p$ , Equation A-3 becomes

$$2l_0\delta = 2Dl_d\theta_p \sin \varphi \quad (\text{A-4})$$

For a special case where the depth of the truss at the face of the column is chosen to be twice the depth of the truss at mid span ( $D=2D_o$ ),  $l_o$  equals  $l_d$ , Equation A-4 yields

$$\delta = \theta_p D \sin \varphi \quad (\text{A-5})$$

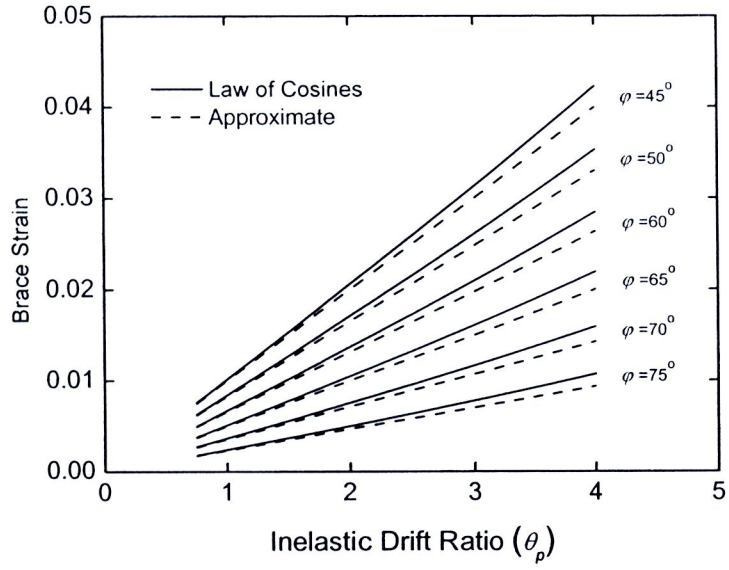
Plastic strain is approximately

$$\varepsilon_p = \frac{\delta}{l_0} \quad (\text{A-6})$$

Substitute Equation A-5 into A-6, gives

$$\varepsilon_p = \frac{\theta_p D \sin \varphi}{l_0} \quad (\text{A-7})$$

The strain values computed from the approximate equation and the law of cosines for the archetype frame are shown for comparison in Figure A.2



**Figure A.2** Brace Strain Demand from Law of Cosine versus Approximation



## **APPENDIX B**

Load Definitions for the Archetype Structure

## B.1 Gravity Loading Definitions

The following gravity loads are assumed for the design of archetype structure. The loads are the same as used by Goel and Chao (2008).

### Dead Loads

- Typical floor (level 2, 3, 4) = 90 psf
- Roof = 85 psf

### Live Loads

- Typical Floor (level 2, 3, 4) = 50 psf
- Roof = 20 psf

### Building Envelope

- Floor slab envelope (for dead load calculations) = 182×122 ft.
- Floor slab envelope (for live load calculations) = 182×122 ft.

### Dead Loads (due to exterior curtain wall)

- At the roof level with 10'-0" tributary height:  $W_{wall}$  = 250 plf
- At levels 3 and 4 with 13'-0" tributary height:  $W_{wall}$  = 325 plf
- At level 2 with 13'-6" tributary height:  $W_{wall}$  = 338 plf

### Floor Seismic Dead Weight (full structure)

- Roof =  $(101.9 \times 180 \times 120) / 1000$  = 2201 kips
- Level 4 =  $(98.5 \times 180 \times 120) / 1000$  = 2128 kips
- Level 3 =  $(99.4 \times 180 \times 120) / 1000$  = 2147 kips
- Level 2 =  $(99.7 \times 180 \times 120) / 1000$  = 2155 kips
- Full structure = 8631 kips

### Beam Load Calculations (for exterior moment frame)

The exterior beams take the dead load from 16 feet of slab (which accounts for 1 foot of overhang). The live load is calculated based on a 15-foot tributary width only.

### Uniformly distributed loads on the exterior beams:

- Roof
  - From slab (dead load) =  $0.085 \times 16$  = 1.36 kip/ft
  - From exterior wall (dead load) = 0.25 kip/ft
  - From slab (live load) =  $0.020 \times 15$  = 0.30 kip/ft
- Level 3, 4
  - From slab (dead load) =  $0.090 \times 16$  = 1.44 kip/ft
  - From exterior wall (dead load) = 0.325 kip/ft
  - From slab (live load) =  $0.050 \times 15$  = 0.75 kip/ft
- Level 2
  - From slab (dead load) =  $0.090 \times 16$  = 1.44 kip/ft
  - From exterior wall (dead load) = 0.338 kip/ft
  - From slab (live load) =  $0.050 \times 15$  = 0.75 kip/ft

### Concentrated Loads at Column Lines (values for one exterior moment frame)

These loads are due to the transverse beams in the E-W direction as well as the exterior wall.

- Roof
 

Dead load (exterior column lines) = $16 \times 4.75 \times 0.085 + 2 \times 16 \times 10 \times 0.025$	= 14.46 kips
Live load (exterior column lines) = $15 \times 3.75 \times 0.020$	= 1.125 kips
Dead load (interior column lines) = $0 + 30 \times 10 \times 0.025$	= 7.5 kips
Live load (interior column lines)	= 0 kips
- Level 3, 4
 

Dead load (exterior column lines) = $16 \times 4.75 \times 0.090 + 2 \times 16 \times 13 \times 0.025$	= 17.24 kips
Live load (exterior column lines) = $15 \times 3.75 \times 0.050$	= 2.8125 kips
Dead load (interior column lines) = $0 + 30 \times 13 \times 0.025$	= 9.75 kips
Live load (interior column lines)	= 0 kips
- Level 2
 

Dead load (exterior column lines) = $16 \times 4.75 \times 0.090 + 2 \times 16 \times 13.5 \times 0.025$	= 17.64 kips
Live load (exterior column lines) = $15 \times 3.75 \times 0.050$	= 2.8125 kips
Dead load (interior column lines) = $0 + 30 \times 13.5 \times 0.025$	= 10.125 kips
Live load (interior column lines)	= 0 kips

### Load Combination (gravity load only)

Loads on beams and columns are calculated based on ASCE7:

$$(1.2 + 0.2S_{DS}) DL + 0.5LL$$

Live load is reducible and assumed 60% of its value.  $S_{DS} = 1.0g$ , therefore:

$$(1.2 + 0.2 \times 1) DL + 0.5 \times 0.6LL = 1.4DL + 0.3LL$$

- Summary of gravity loading (pattern loading is not accounted for)
 

$w_1$ (level 2) = $1.4(1.44 + 0.338) + 0.3(0.75)$	= 2.71 kip/ft
$w_2$ (level 3, 4) = $1.4(1.44 + 0.325) + 0.3(0.75)$	= 2.70 kip/ft
$w_3$ (roof) = $1.4(1.36 + 0.25) + 0.3(0.3)$	= 2.34 kip/ft
$L_1$ (level 2) = $1.4(17.64) + 0.3(2.8125)$	= 25.54 kips
$L_2$ (level 2) = $1.4(10.125) + 0.3(0)$	= 14.18 kips
$L_3$ (level 3, 4) = $1.4(17.24) + 0.3(2.8125)$	= 24.98 kips
$L_4$ (level 3, 4) = $1.4(9.75) + 0.3(0)$	= 13.65 kips
$L_5$ (roof) = $1.4(14.46) + 0.3(1.125)$	= 20.58 kips
$L_6$ (roof) = $1.4(7.5) + 0.3(0)$	= 10.5 kips

The distribution of gravity loads are shown in Figure B.1.

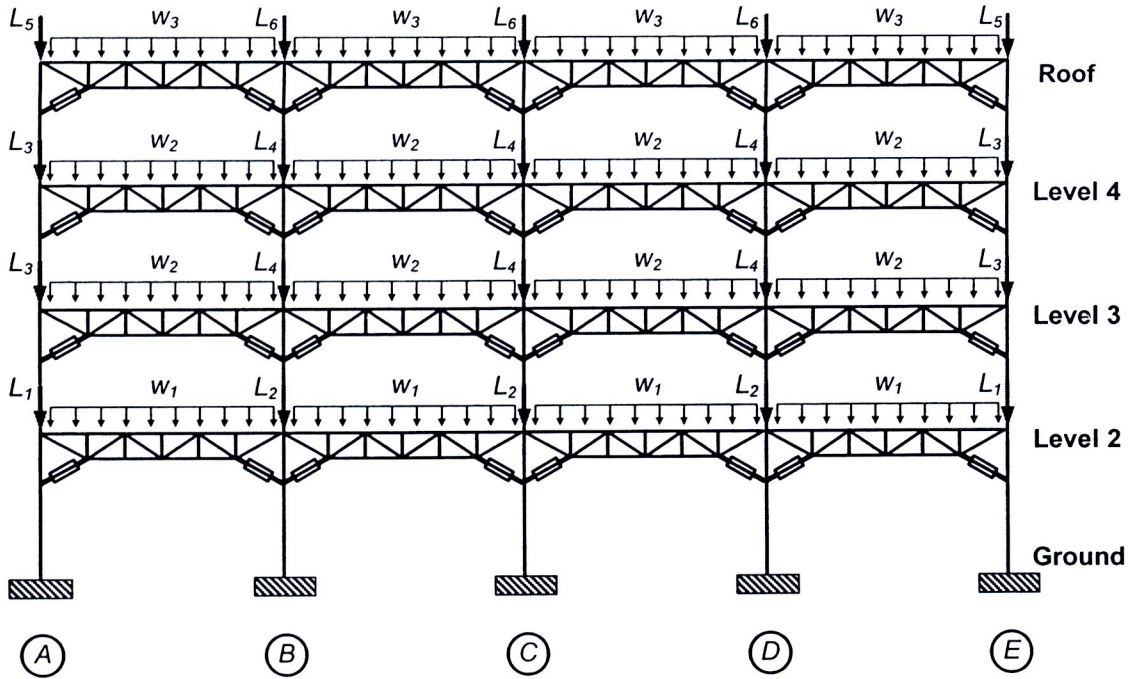


Figure B.1 Gravity Load Definition

## B.2 Design Base Shear and Lateral Force Distribution

The design base shear was determined for two levels of performance objectives: 1) a 2% maximum story drift ratio ( $\theta_u$ ) for a ground motion hazard with a 10% probability of exceedance in 50 years (10/50 or 2/3MCE), and 2) a 3% maximum story drift ratio ( $\theta_u$ ) for a 2% probability of exceedance in 50 years (2/50 or MCE). The design response spectrum for the first hazard level (2/3MCE) is determined as follows (ASCE, 2010):

Acceleration parameter

$$S_s = 1.5g \text{ and } S_l = 0.6g$$

According to soil profile type D and acceleration parameter  $S_s$  and  $S_l$ , therefore,

$$F_a = 1 \text{ and } F_v = 1.5$$

The  $MCE_R$  spectral response acceleration parameter for short periods ( $S_{MS}$ ) and at 1 s ( $S_{M1}$ ), can be determined

$$S_{MS} = F_a S_s = 1(1.5g) = 1.5g$$

$$S_{M1} = F_v S_l = 1.5(0.6g) = 0.9g$$

Estimated period,  $T$ , 0.94 sec is greater than  $T_s = \frac{S_{D1}}{S_{DS}} = \frac{S_{M1}}{S_{MS}} = 0.6 \text{ sec}$

Therefore, spectral acceleration at MCE level is

$$S_a = \frac{S_{M1}}{T} = \frac{0.9}{0.94} = 0.96g$$

For 2/3MCE level

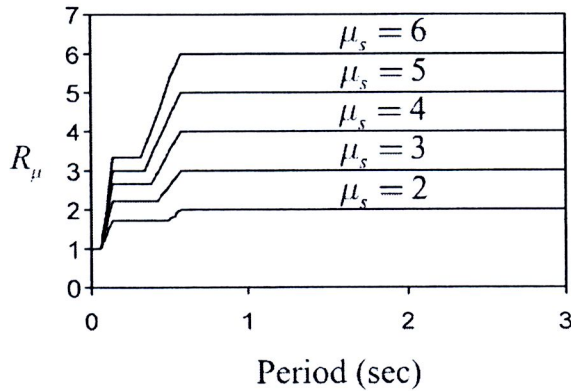
$$S_a = \frac{2}{3} \times 0.96g = 0.64g$$

A yield drift ratio ( $\theta_y$ ) of 0.75% is assumed. The ductility reduction factor,  $R_\mu$  (see Figure 2.8), is obtained from idealized inelastic spectra from Newmark and Hall (1982). The  $R_\mu$ - $\mu$ - $T$  relationship is shown in Figure B.2 and Table B.1 (Miranda and Bertero, 1994; Lee and Goel, 2001).

**Table B.1**  $R_\mu$ - $\mu$ - $T$  relationship by Newmark and Hall (1982)

Period Range	Ductility Reduction Factor
$0 \leq T \leq \frac{T_1}{10}$	$R_\mu = 1$
$\frac{T_1}{10} \leq T \leq \frac{T_1}{4}$	$R_\mu = \sqrt{2\mu_s - 1} \left( \frac{T_1}{4T} \right)^{2.513 \log \left( \frac{1}{\sqrt{2\mu_s - 1}} \right)}$
$\frac{T_1}{4} \leq T \leq T_1'$	$R_\mu = \sqrt{2\mu_s - 1}$
$T_1' \leq T \leq T_1$	$R_\mu = \frac{T\mu_s}{T_1}$
$T_1 \leq T$	$R_\mu = \mu_s$

Note:  $T_1 = 0.57$  sec,  $T_1' = T_1 \sqrt{2\mu_s - 1} / \mu_s$  sec.



**Figure B.2**  $R_\mu$ - $\mu$ - $T$  relationship by Newmark and Hall (1982) for EP-SDOF

The period of the archetype structure is 0.94 sec, the ductility reduction factor,  $R_\mu$ , equals to  $\mu_s$ . The energy modification factor,  $\gamma$ , is determined by Equation (2-2).

$$\mu_s = \theta_u / \theta_y = 2 / 0.75 = 2.67$$

$$\gamma = \frac{2\mu_s - 1}{R_\mu^2} = \frac{2(2.67) - 1}{2.67^2} = 0.61$$

The lateral force distribution factor,  $\lambda_i$ , and the story shear ratio,  $\beta_i$ , can be computed by using Equations (2-6) and (2-7). Then the parameter  $\alpha$  is determined from Equation (2-4)

$$\alpha = \left( \sum_{j=1}^n \lambda_j h_j \right) \frac{\theta_p 8\pi^2}{T^2 g} = 2.939 \frac{0.02 \times 8\pi^2}{0.94^2 \times 32.2} = 1.472$$

Therefore, for 2/3MCE level

$$\frac{V}{W} = \frac{-\alpha + \sqrt{\alpha^2 - 4\gamma C_e^2}}{2} = \frac{-1.472 + \sqrt{1.472^2 - 4(0.61)0.64^2}}{2} = 0.154$$

$$V = 0.154W = 0.154(8631) = 1325 \text{ kips (5893 kN)}$$

The 2/3MCE level governs the design. Hence, the design base shear is 1325 kips (5893 kN), and the lateral force distribution is shown in Table B.2.

**Table B.2** Lateral Force Distribution of the Archetype Structure

Floor	$h_i$ (ft)	$w_i$ (kips)	$w_i h_i$	$\sum w_i h_i$	$\beta_i$	$\beta_i - \beta_{i+1}$	$(\beta_i - \beta_{i+1})h$	$F_i$ (kips)
Roof	53	2201	116653	116653	1.000	1.000	53.00	664
4	40	2128	85120	201773	1.516	0.516	20.65	343
3	27	2147	57969	259742	1.837	0.321	8.66	213
2	14	2155	30170	289912	1.997	0.160	2.24	106

The calculated values of all significant parameters are listed in Table 4.3; the design base shear for the first hazard level (2/3MCE) governs the design. In addition  $M_{pc}$  is calculated by

$$M_{pc} = \frac{1.1V'h_1}{4} = \frac{1.1 \times \left( \frac{1325}{4} \right) \times 14}{4} = 638 \text{ k.ft (865 kN.m)}$$

## **APPENDIX C**

### Example of Design Calculation

### C.1 Analysis of Truss

To calculate required strength of the truss, a superposition method or a computer analysis can be used because all truss members are designed to remain elastic. As an example, the structural analysis results of the truss at the 3<sup>rd</sup> floor level subjected to the gravity loads and adjusted BRB forces (Figure C.1) are given in Table C.1. For this example, the effects due to the plastic moments of the chords were computed using the superposition method.

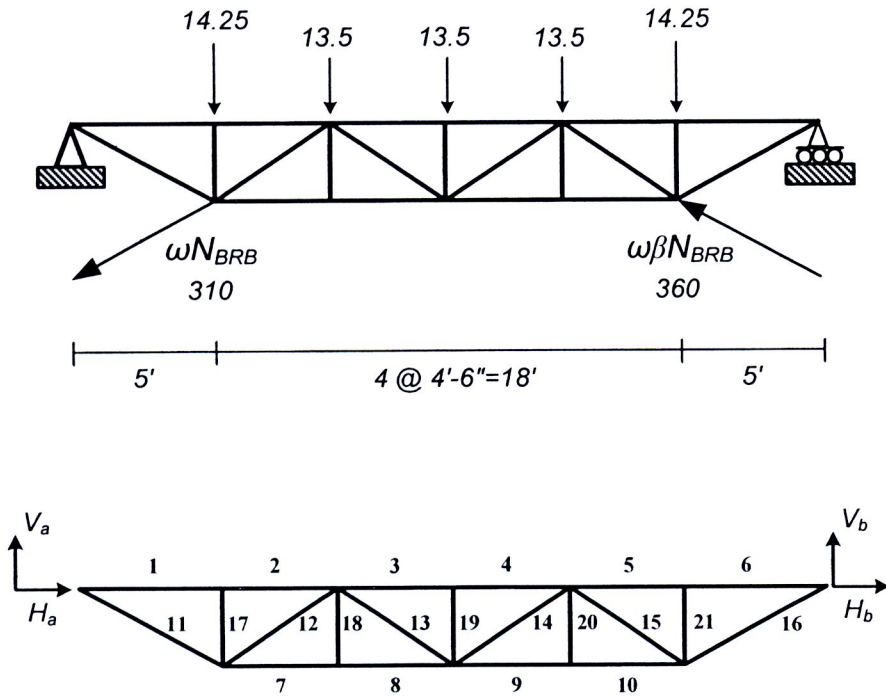


Figure C.1 Structural Analysis of the Truss at 3<sup>rd</sup> Floor

**Table C.1** Analysis Result of the Truss at 3<sup>rd</sup> Floor

Member	Due to Point Loads (kips)	Due to BRB Forces (kips)	Combination (kips)	Required Strength (kips)
$V_a$ (reaction)	35	32	66	
$V_b$ (reaction)	35	54	89	
$H_a$ (reaction)	0	322	322	
$H_b$ (reaction)	0	277	277	
1	-69	-385	-454	
2	-69	-385	-454	
3	-118	0	-118	
4	-118	0	-118	
5	-69	385	316	385/-454
6	-69	385	316	
7	105	148	253	
8	105	148	253	
9	105	-237	-132	
10	105	-237	-132	
11	77	71	148	
12	-42	220	179	
13	14	-220	-206	234/-262
14	14	220	234	
15	-42	-220	-262	
16	77	-121	-44	
17	-14.25	0	-14.25	
18	0	0	0	
19	-13.50	0	-13.50	14.25/-14.25
20	0	0	0	
21	14.25	0	14.25	

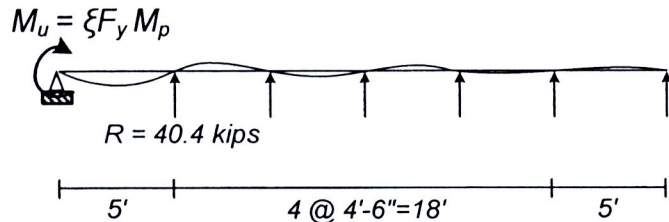
Note: Positive reaction is upward or to the right and positive force in truss is tension.

## C.2 Section Design of Truss

Location: Chord at 3<sup>rd</sup> floor level

- Required chord compressive strength = 454 kips
- Section = 2MC7x19.1
- Separation = 3/8 in (connected at every  $l/3$ )
- $r_x = 2.77; r_y = 1.64$
- $kl/r_x = 21.65; kl/r_y = 36.6$
- $0.75kl/r_y = 27.45 > ka/r = 19.24$  (Built-up section requirement)
- AISC Equation 6-2 (a);  $(kl/r)_m = 36.6 < 4.71\sqrt{E/F_y} = 113.43$
- $F_{cr} = \left[ 0.658 \frac{F_y}{F_c} \right] F_y = 45.33 \text{ ksi}$
- Compressive strength of chord  
 $\phi_c P_n = \phi_c F_{cr} A_g = 458 \text{ kips} (\phi_c = 0.9, A_g = 11.22 \text{ in}^2) > 454 \text{ kips}$
- Tension strength of chord  
 $\phi_t P_n = \phi_t F_y A_g = 504 \text{ kips} (\phi_t = 0.9, A_g = 11.22 \text{ in}^2) > 385 \text{ kips}$

The analysis results in Table C.1 are due to gravity loads and the adjusted BRBs forces only. The additional axial loads in the first vertical members caused by external moments due to plastic hinges in the top chords can be computed by treating the top chord as a continuous beam as shown in Figure C.2



**Figure C.2** Multi-span Beam Subjected to an End Moment

- Section of chord = 2MC7x19.1
- Plastic modulus = 28.6 in<sup>3</sup>
- Total applied moment:  
 $M_u = \xi M_p = \xi F_y Z_x = 1.2(50)(28.3) = 1698 \text{ k}\cdot\text{in}$
- Force in the first vertical member  
 $F = M_u / 3.5 = 141.5 / 3.5 = 40.4 \text{ kips}$
- Total force of in vertical member  
 $14.25 + 40.4 = 54.65 \text{ kips}$
- Axial strength of A36 L3.5x3.5x5/16 = 58 kips

Therefore, L3.5x3.5x5/16 is acceptable. Because the member might also be subjected to bending moment, 2L3.5x3.5x5/16 are provided to avoid any out-of-plane determination.

### C.3 Analysis of Column

After the lateral forces required for equilibrium are calculated as shown in Table 4.9, the design moments and axial forces for the column in each story are computed by analyzing the cantilever column tree. Figure C.3 shows an example of a column tree analysis of the interior column subjected to forces acting to the right.

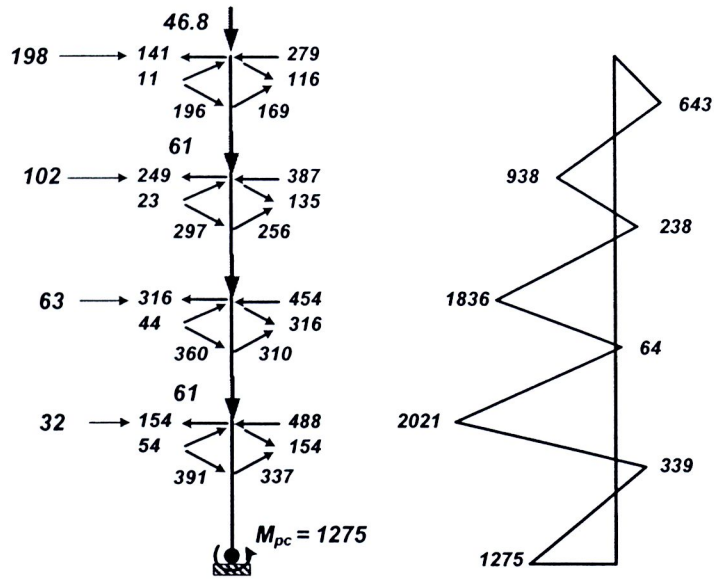


Figure C.3 Column Tree Analysis

## C.4 Section Design of Column

Location: Interior Column 3<sup>rd</sup> floor level

- Design column moment = 1836 kip.ft
- Design axial force = 182 kips
- Equivalent Inertia of Truss girder at fourth level = 4311 in<sup>4</sup> (approximate)
- Equivalent Inertia of Truss girder at third level = 5049 in<sup>4</sup> (approximate)
- Column at third level = W24x192 ( $I = 6260$  in<sup>4</sup>)
- Column at second level = W24x229 ( $I = 7650$  in<sup>4</sup>)

- Shear Strength of Column

$$\phi_v V_n = \phi_v 0.6 F_y A_w C_v = 620 \text{ kips} \quad (\phi_v = 1.0, A_w = 20.7, C_v = 1.0 \text{ in}^2)$$

$$\phi_v V_n = 620 > V_u = 198 + 102$$

$$\bullet \quad G_a = \frac{\sum (EI/L)_c}{\sum (EI/L)_g} = \frac{6260/13 + 6260/13}{4311/30 + 4311/30} = 3.35$$

$$\bullet \quad G_b = \frac{6260/13 + 7650/13}{5049/30 + 5049/30} = 3.18$$

- $k_x = 1.9$  (based on Alignment Chart-sideways uninhibited)

- $k_y = 1.0$  (assuming the column is laterally braced in y-direction)

- $r_x = 10.54$  in;  $r_y = 3.07$  in (W24x192)

- $k_x l / r_x = 20.03 < k_y l / r_y = 31.29 \Rightarrow$  The minor axis controls  $\phi_c F_{cr}$

$$\bullet \quad kl/r = 31.29 \leq 4.71 \sqrt{E/F_y} = 4.71 \sqrt{29000/50} = 113.43$$

$$\bullet \quad F_{cr} = \left[ 0.658 \frac{F_y}{F_c} \right] F_y = 46.87 \text{ ksi}$$

- The design compressive strength of column ( $L_b \approx 8\text{ft.}$ ):

$$\phi_c P_n = \phi_c F_{cr} A_g = 2375 \text{ kips } (\phi = 0.9, A_g = 56.3 \text{ in}^2)$$

The design flexural strength:

$$\phi M_n = \phi_b F_y Z_x = 2100 \text{ kip.ft } (\text{Note } L_p \text{ for W24x192 is } 8\text{ft.})$$

For  $P_u / \phi_c P_n = 0.05 < 0.2$ , the member strength must satisfy

$$\frac{P_r}{2\phi_c P_n} + \left( \frac{M_{rx}}{\phi_b M_{nx}} \right) \leq 1.0$$

where

$$M_r = B_1 M_{nt} + B_2 M_{lt}$$

$$P_r = P_{nt} + B_2 P_{lt}$$



For plastic design, it is conservative to determine  $M_{rx}$  based on  $B_2 \times$  Total Moment, where Total Moment = non-sway moment + sway moment (Salmon et al., 2009).

$$B_2 = \frac{1}{1 - \frac{\alpha P_{story}}{P_{e story}}} \geq 1$$

where  $\alpha = 1.0$  (LRFD),  $P_{story}$  = total vertical loads supported by the story, including gravity column load,  $H$  = story shear

$$P_{e story} = R_M \frac{HL}{\Delta_H}$$

where  $R_M = 1 - 0.15(P_{mf} / P_{story})$ ,  $P_{mf}$  = total vertical loads in column that are a part of moment frames, hence:

$$R_M = 1 - 0.15(1/3) = 0.95$$

$$P_{e story} = R_M \frac{HL}{\Delta_H} = R_M \frac{H}{\Delta_H / L} = 0.95 \frac{1006}{0.02} = 47785$$

$$B_2 = \frac{1}{1 - \frac{\alpha P_{story}}{P_{e story}}} = \frac{1}{1 - \frac{(1)4329}{47785}} = 1.1$$

$$\frac{B_2 P_r}{2\phi_c P_n} + \left( \frac{B_2 M_{rx}}{\phi_b M_{nx}} \right) = \frac{1.1(182)}{2(2375)} + \left( \frac{1.1(1837)}{2100} \right) = 1.00$$

Therefore W24x192 is acceptable.

## **APPENDIX D**

### Determination of Fragility Curves

The data from incremental dynamic analysis (IDA) including drifts and a collapse state are used to create fragility curves. The probability for different response states are assumed to follow log-normal distribution functions. The discrete data points are fitted using the least square method. The process is described briefly below.

### D.1 Logarithmic Normal Distribution

A random variable  $X$  has a *logarithmic normal* (or simply log-normal) probability distribution if  $\ln X$  is normal. In this case, the probability density function,  $f_X(x)$ , of  $X$  is

$$f_X(x) = \frac{1}{x\zeta\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{\ln x - \lambda}{\zeta}\right)^2\right], \quad 0 < x < \infty \quad (\text{D-1})$$

where  $\lambda$  and  $\zeta$  are the parameters.

The mean  $\mu$  and variance  $\sigma^2$  for log-normal variable are obtained as:

$$\mu = e^{\left(\lambda + \frac{1}{2}\zeta^2\right)} \text{ and } \sigma^2 = e^{2\lambda + \zeta^2} (e^{\zeta^2} - 1) \quad (\text{D-2})$$

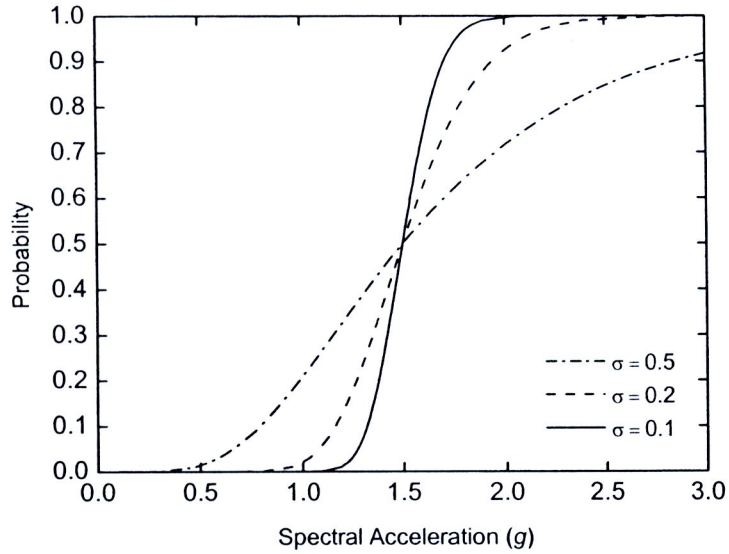
In terms of  $\mu$  and  $\delta = \sigma/\mu$  (coefficient of variation), the parameters  $\lambda$  and  $\zeta$  are obtained as:

$$\lambda = \ln \frac{\mu}{\sqrt{1 + \delta^2}} \text{ and } \zeta = \sqrt{\ln(1 + \delta^2)} \quad (\text{D-3})$$

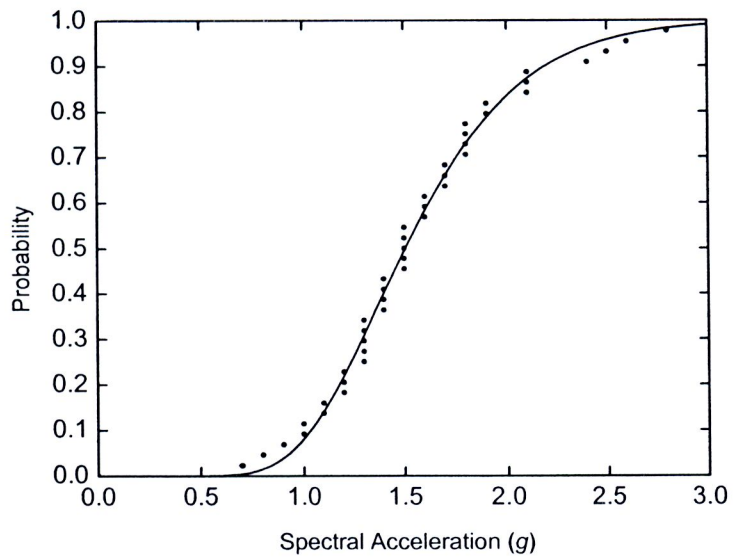
The cumulative distribution function of a log-normal variable  $X$  is given by:

$$\begin{aligned} F_X(x) &= \Phi\left(\frac{\ln x - \lambda}{\zeta}\right) \\ &= \frac{1}{2} \left[ 1 + \operatorname{erf}\left(\frac{1}{\sqrt{2}} \frac{\ln x - \lambda}{\zeta}\right) \right], \quad 0 < x < \infty \end{aligned} \quad (\text{D-4})$$

where *erf* is the error function. Examples of a cumulative distribution function for a different value of  $\sigma$  with  $\mu$  equals to 1.5 are shown in Figure D.1. Collapse probability from IDA results can be fitted by the least square method as shown in Figure D.2.

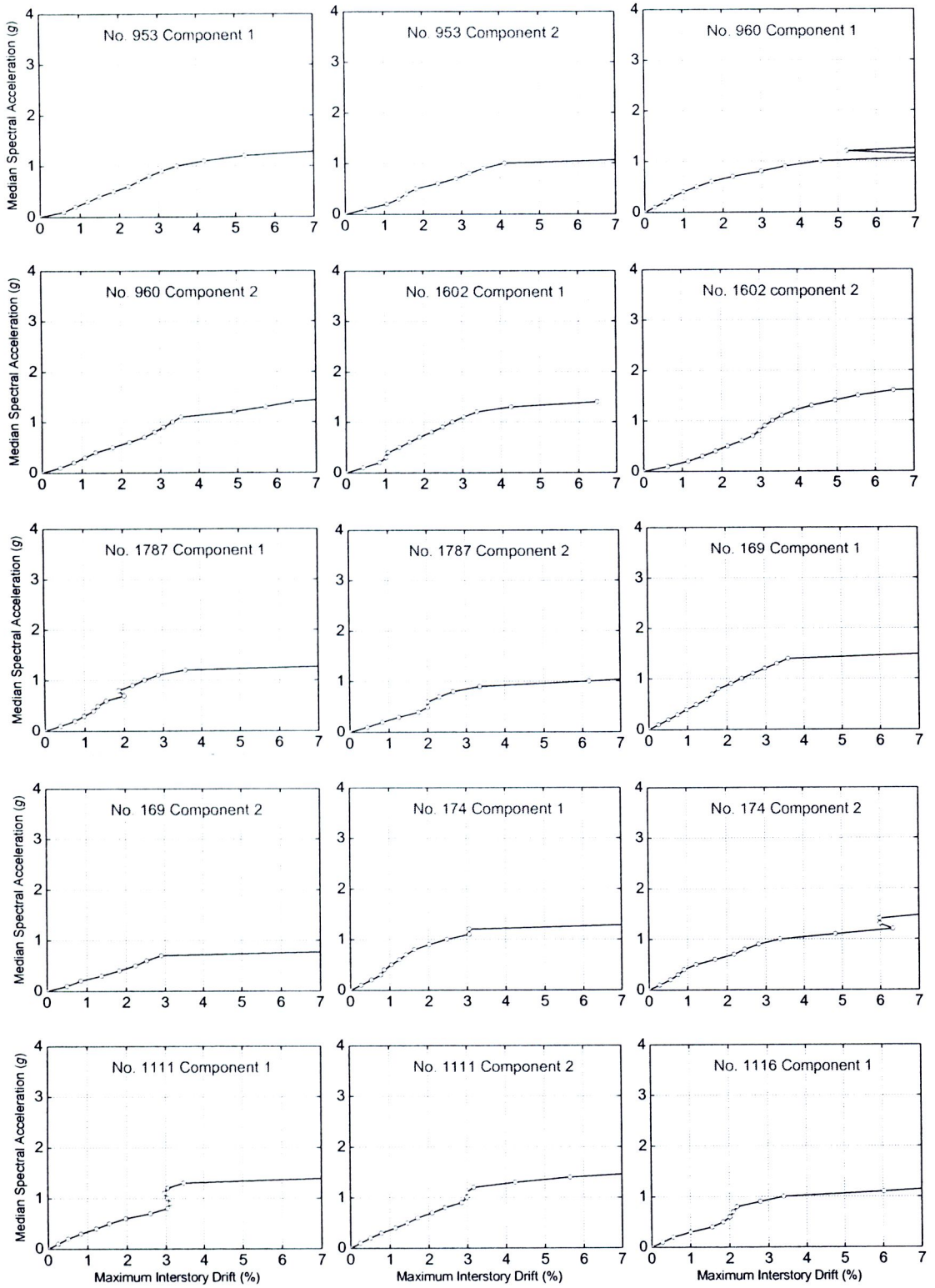


**Figure D.1** Examples of Cumulative Distribution Functions

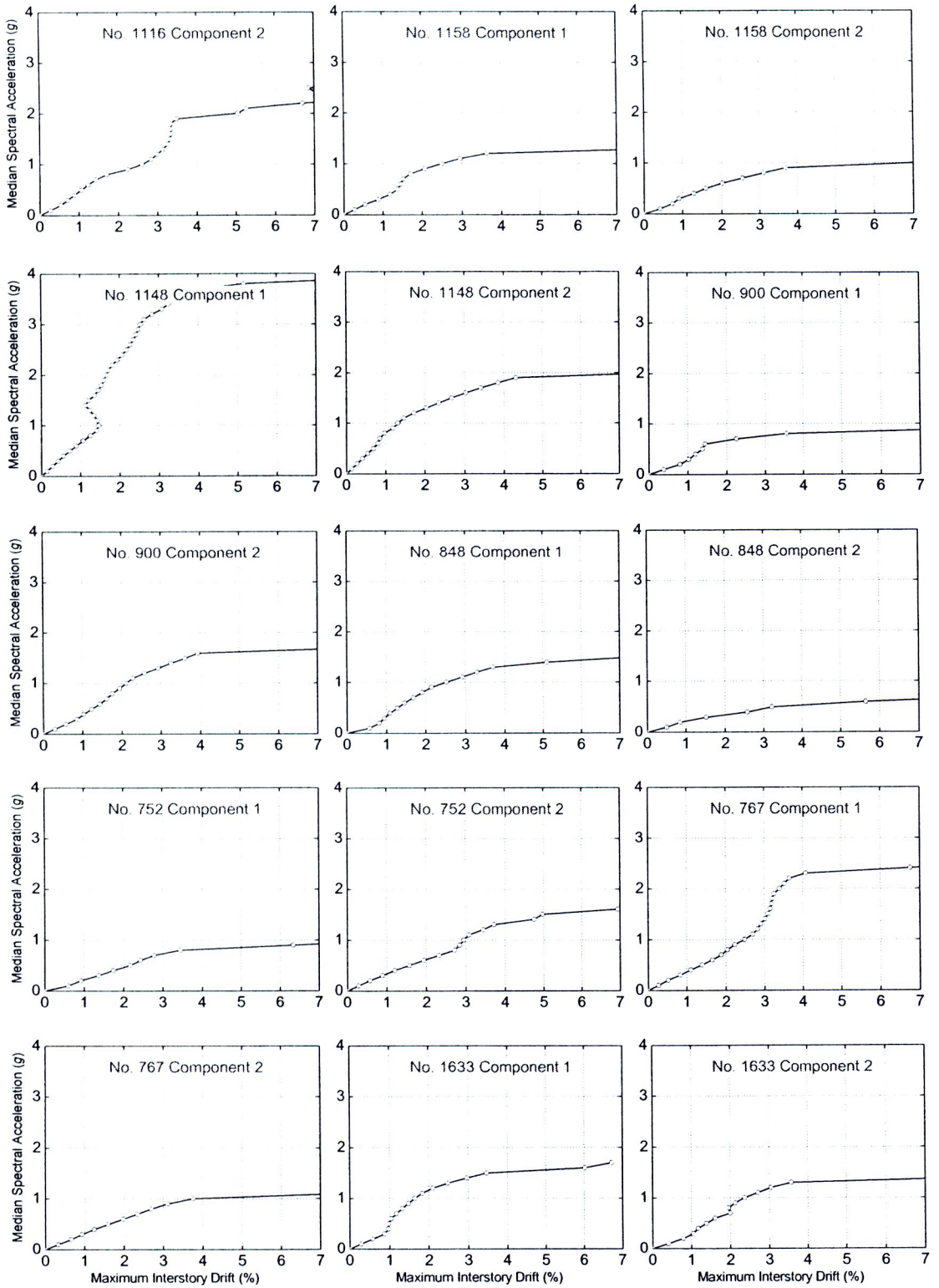


**Figure D.2** Least Square Fit for the Probability of Collapse (Fragility Curve)

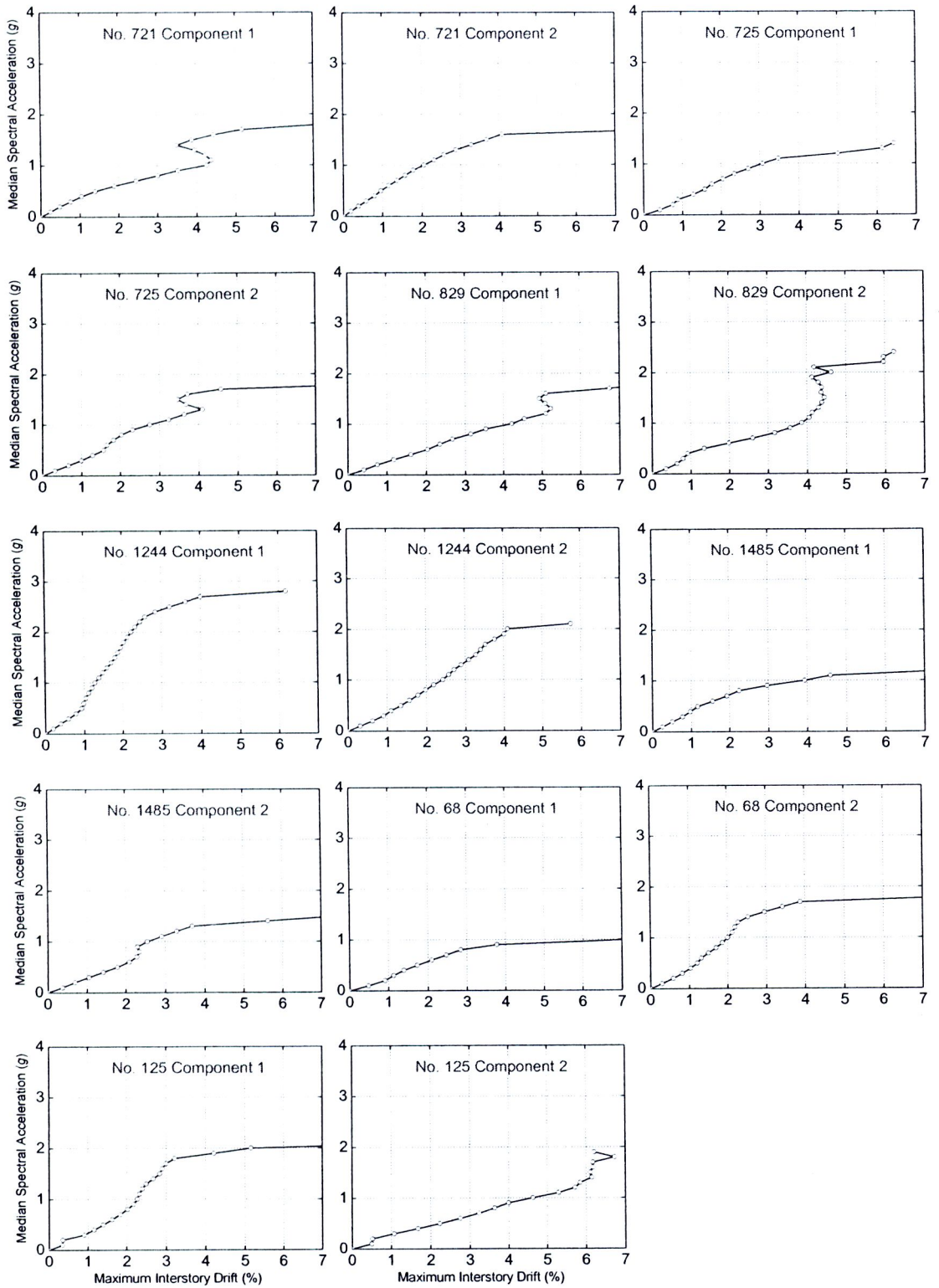
In Figure D.2, each data point is obtained from calculating probability of collapse at a given  $S_a$  value using the IDA results based on 44 ground motions. The IDA results are shown in Figure D.3



**Figure D.3** IDA Results of Each Ground Motion Component



**Figure D.3 IDA Results of Each Ground Motion Component (Continued)**



**Figure D.3** IDA Results of Each Ground Motion Component (Continued)

## CURRICULUM VITAE

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<b>PUBLICATION</b>	Wongpakdee, N. and Leelataviwat, S., 2011, "Collapse Evaluation Using Incremental Dynamic Analysis Based on FEMA P695 Methodology", <b>The 17<sup>th</sup> National Convention on Civil Engineering</b> , 9-11 May, 2011, Kasetsart University, Thailand.

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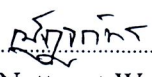
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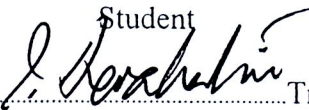
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