2D Simple Analysis of Van Nuys Hotel

1. Used Tools

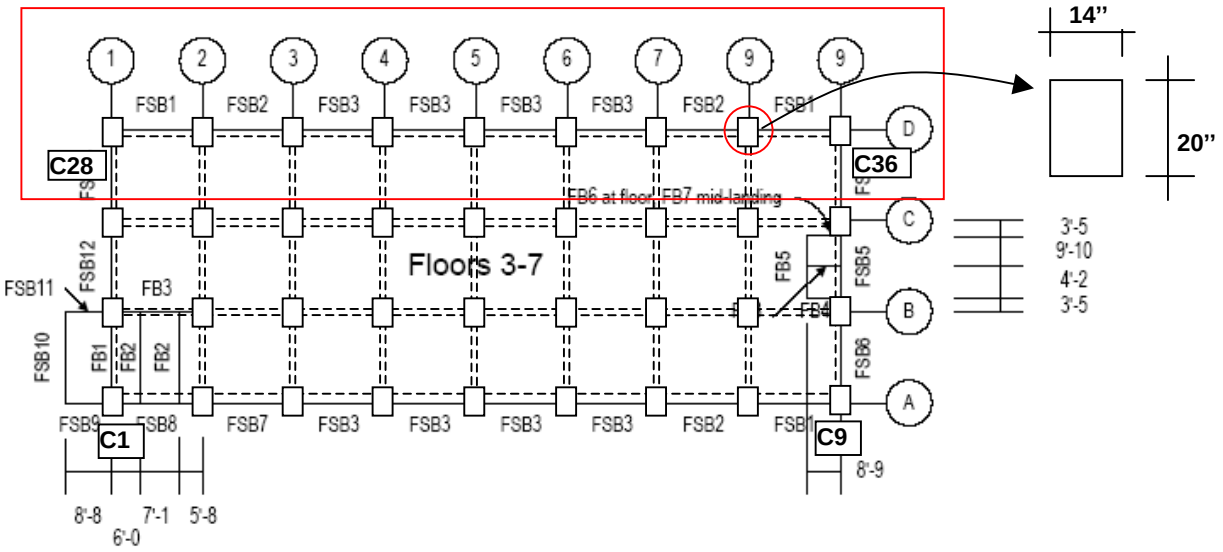
   -. OpenSees : Determine Moment, Shear and Axial force Demand

   -. FEDEASLab : Determine Moment Capacity of Beams and P-M interaction of Columns

   -. ACI318 : Determine Shear Capacity of beams and columns

   -. UBC64(1964) : Estimate Lateral Load

2. Target Frame and Design Summary



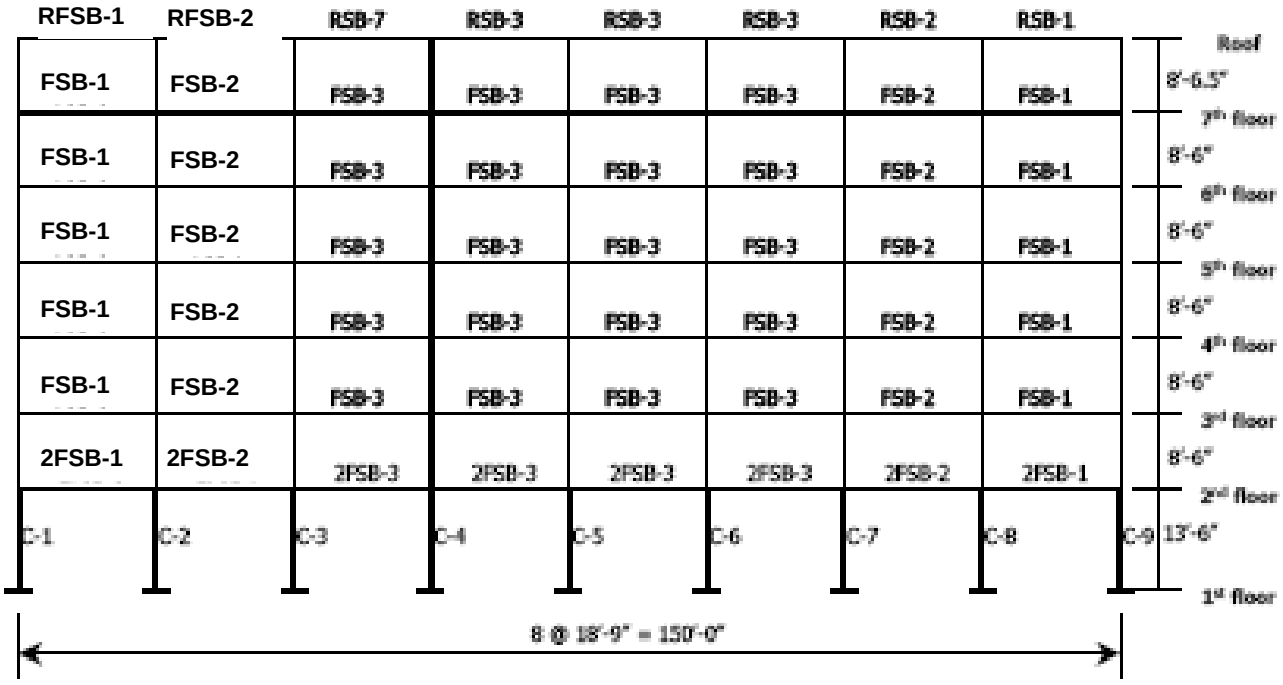


Table 2.1 Column reinforcement schedule

		Column mark							
		C-13 to C-17, C-21 to C-26	C-11, C-12, C-20	C-30 to C-34	C-10, C-18, C-19, C-27	C-2, C-3, C-8, C-29, C-35	C-1, C-9, C-28, C-36	C-1A, C-10A	C-17A, C-26A
Level	Col size	18"x18"	18"x18"	14"x20"	14"x20"	14"x20"	14"x20"	10"x12"	10"x12"
7th floor	Vert. bars	6-#7	6-#7	6-#7	6-#7	6-#7	6-#7	4-#5	
	Ties	#2@12"	#2@12"	#2@12"	#2@12"	#2@12"	#2@12"	#2@10"	
6th floor	Vert. bars	6-#7	6-#7	6-#7	6-#7	6-#7	6-#7	4-#5	4-#5
	Ties	#2@12"	#2@12"	#2@12"	#2@12"	#2@12"	#2@12"	#2@10"	#2@10"
5th floor	Vert. bars	6-#7	6-#8	6-#7	6-#7	6-#7	6-#7	4-#5	4-#5
	Ties	#2@12"	#3@12"	#2@12"	#2@12"	#2@12"	#2@12"	#2@10"	#2@10"
4th floor	Vert. bars	6-#8	8-#9	6-#7	6-#9	6-#7	6-#7	4-#5	4-#5
	Ties	#3@12"	#3@12"	#2@12"	#3@12"	#2@12"	#2@12"	#2@10"	#2@10"
3rd floor	Vert. bars	8-#9	12-#9	6-#9	8-#9	8-#9	6-#7	4-#6	4-#5
	Ties	#3@12"	#3@12"	#3@12"	#3@12"	#3@12"	#2@12"	#2@10"	#2@10"
2nd floor	Vert. bars	10-#9	12-#9	6-#9	8-#9	8-#9	6-#7	4-#6	4-#5
	Ties	#3@12"	#3@12"	#3@12"	#3@12"	#3@12"	#2@12"	#2@10"	#2@10"
1st floor	Col size	20"x20"	20"x20"						
	Vert. bars	10-#9	12-#9	10-#9	12-#9	10-#9	8-#9	4-#8	4-#6
	Ties	#3@12"	#3@12"	#3@12"	#3@12"	#3@12"	#3@12"	#3@10"	#2@10"

Table 2.2 Spandrel beam reinforcement schedule, floors 3 through 7

Beam mark	Width	Height	Top bars					Bottom bars	#3 ties
			7F	6F	5F	4F	3F		
FSB-1	16″	22-½″	①⑨ 2#7	2#9	2#9	3#8	3#8	2#7 (2#8 @ 3F, 4F)	①⑨ 3@5″, 5@6″, rest @10″, 3F- 5F
			②⑧ FSB-2 top bars						②⑧ 6@4″, 5@6″, 3F-5F
FSB-2	16″	22-½″	②⑧ 2#9	3#8	3#8	3#8	3#9	2#6	8@5″, 5@6″ ea end
			③⑦ FSB-3 top bars						Rest @ 10″ 3F-5F
FSB-3	16″	22-½″	2#8	2#9	3#8	3#8	3#9	2#6	3@5″, 5@6″ ea end
									Rest @ 10″ 3F-5F
FSB-7	16″	22-½″	③ FSB-3 top bars					2#7	3@5″, 5@6″ ea end
			② FSB-8 top bars						Rest @ 10″ 3F-5F
FSB-8	16″	22-½″	② 2#8	2#9	2#9	3#8	3#8	2#7 (2#8 @ 5F, 2#9 @ 3F, 4F)	① 3@5″, 5@6″, rest@10″ 3F-5F
			① 2#7	2#8	2#9	2#9	3#8		② 6@4″, 5@6″ 3F-5F

①, ②, etc.: column lines
3F, 4F, etc: floor levels

Table 2.3 Roof and second-floor spandrel beam reinforcement schedule

Beam mark	Width	Height	Top bars	Bottom bars	#3 ties
RSB-1	16″	22″	①⑨ 2#6 ②⑧ 2#8	2#7	#3@10″
RSB-2	16″	22″	②⑧ RSB-1 top bars ③⑦ RSB-3 top bars	2#6	Same
RSB-3	16″	22″	2#8	2#6	Same
RSB-7	16″	22″	④ RSB-3 top bars ③ 2#9	2#6	Same
RSB-8	16″	22″	③ 2#9 ② 3#9	2#9	Same
2FSB-1	16″	30″	①⑨ 2#9 ②⑧ 2FSB-2 top bars	2#8	4 @ 6″, 2 @ 8″, ea end, rest @ 13″
2FSB-2	16″	30″	②⑧ 3#8 ③⑦ 2FSB-3 top bars	2#6	Same
2FSB-3	16″	30″	2#9	2#6	Same
2FSB-7	16″	30″	③ 2FSB-3 top bars ② 2FSB-8 top bars	2#7	Same
2FSB-8	16″	30″	② 2#9 ① 2#9	2#8	Same

3. Gravity and Lateral Loads

-. Building Weight is estimated based on the column dead loads of Van Nuys Database.

Gravity loads - DL + 10 PSF partition load

Floor	column		
	Interior	Exterior	Corner
7	41	29.8	20.2
6	88	63.8	43.2
5	135	97.8	66.2
4	182	131.8	89.2
3	229	165.8	112.2
2	276	199.8	135.2
1	332.4	237.3	161.1

$$W = 332.4\times 14(No.\ of\ \text{int}.\ columns) + 237.3\times 18(No.\ of\ ext.\ columns) + 161.1\times 4(No.\ of\ cor.\ columns)$$
$$+ 0.150\times(14\times 20/12^2)\times 13.5\times 36 = 9711.15\ \ kips$$

-. The above column dead loads are converted into the distributed gravity load on each beam.

-. Lateral loads are estimated based on UBC64 by which Van Nuys Hotel was designed

-. Factor 1.5 is used to approximately magnify ASD design force

$V = 1.5ZKCW$

; $Z = 1.0$ *for Zone No.*3

$K = 0.67$ *for the building with a moment resisting space frame*

$C = \dfrac{0.05}{\sqrt[3]{T}} = 0.0563$, $T = \dfrac{N}{10}$

$\Rightarrow V_{half} = 1.5\times 0.67\times 0.563\times 9711.15/2 = 274.80 kips$

$\Rightarrow F_x = V_{half}\times \dfrac{w_x h_x}{\sum w_x h_x}$

Roof	1.5X38.73
7F	1.5X38.43
6F	1.5X32.59
5F	1.5X26.76
4F	1.5X20.93
3F	1.5X15.10
2F	1.5X10.67

4. Details of Analysis with OpenSees

-. Concrete Material : “Concrete01” considering unconfined for all

$$\varepsilon_{psc0} = 0.002 \ , \ \varepsilon_{psu} = 0.004, \ f_{pcu} = 0.2 f_{pc}$$

-. Reinforcing steel : “Steel02” with hardening ratio 2%

-. BeamColumn element : “beamWithHinges” with plastic hinge length = depth

-. Effective Moment of Inertia for elastic region : **0.5 I_g for both column and beam**

-. **Ignore the contribution of slab** when modeling beam and calculating flexural moment capacity

-. Only two exterior frames provide total lateral resistance.

-. All load information come from the data posted on the OpenSees Web about Van Nuys Hotel

-. Assume cover = 2.5in(from outer surface to center of reinforcing bars)

-. Without P-M interaction and Shear limit state material

-. Rigid Beam Column Joints

5. Analysis Results

	Moment Demand(Kip-in)		Moment Capacity(Kip-in)		Demand/Capacity	
	Interior	Exterior	Interior	Exterior	Interior	Exterior
2FSB-1	-2612.37	-2658.03	-2301.6 (2524.4)	-1964.9 (2141.2)	1.14	1.35
2FSB-2	-2419.29	-2947.88	-1962.1 (2141.2)	-2299.6 (2524.4)	1.23	1.28
2FSB-3	-2692.82		-1962.1 (2141.2)		1.37	
FSB-1(5F)	-1802.46	-1488.7	-1559.7 (1785.9)	-1335.3 (1521.6)	1.16	1.11
FSB-2(5F)	-1813.31	-1707.11	-1559.7 (1785.9)	-1559.7 (1785.9)	1.16	1.09
FSB-3(5F)	-1784.77		-1559.7 (1785.9)		1.14	
RFSB-1	-752.43	-439.76	-1048.1 (1183.5)	-635.55 (671.2)	0.72	0.69
RFSB-2	-704.44	-726.53	-1046.5 (1183.5)	-1046.5 (1183.5)	0.67	0.69
RFSB-3	-699.53		-1046.5 (1183.5)		0.67	

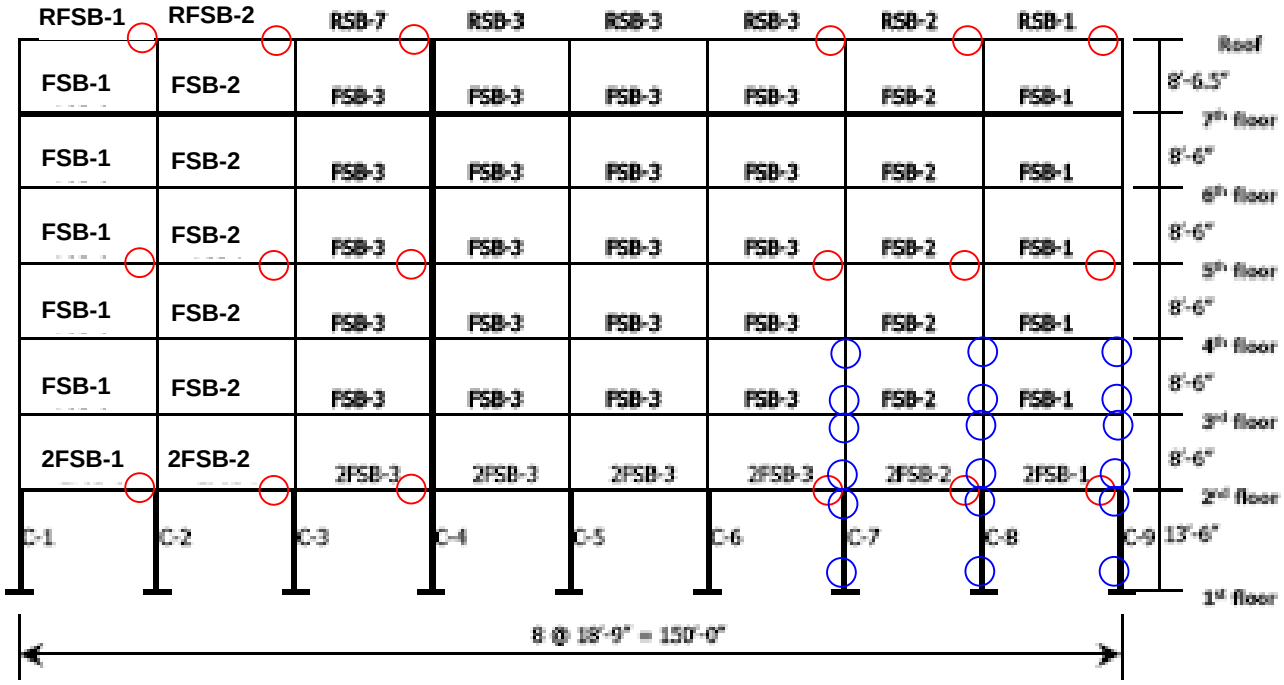
( ) : Moment Capacity by ACI method

	Shear Demand(Kip)		Shear Capacity(Kip)		Demand/Capacity	
	Interior	Exterior	Interior	Interior	Interior	Exterior
2FSB-1	38.32	39.68	81.60		0.47	0.49
2FSB-2	34.84	38.06	81.60		0.43	0.47
2FSB-3	36.04	37.21	81.60		0.44	0.46
FSB-1(5F)	29.18	26.07	70.20	59.71	0.42	0.44
FSB-2(5F)	28.13	27.22	59.71	59.71	0.47	0.46
FSB-3(5F)	27.92	27.39	59.71	59.71	0.47	0.46
RFSB-1	18.25	15.34	44.75		0.41	0.34
RFSB-2	16.78	16.97	44.75		0.37	0.38
RFSB-3	16.80	16.68	44.75		0.38	0.37

	P–M Interaction Demand		Capacity	Demand
	P(Kip)	M_{max}(Kip-in)	M_{max}(Kip-in)	/Capacity
1C-36	227.92	3414.7	3296.58	1.04
1C-35	239.01	3597.06	≈3336.87	1.08
1C-34	235.21	3357.62	≈3323.07	1.01
2C-36	181.08	(698.59)[1]	1972.71	-
2C-35	203.06	1934.54	3227.67	0.60
2C-34	198.48	1513.02	2676.99	0.57
3C-36	145.73	1162.68	1777.76	0.65
3C-35	169.30	1846.61	3042.73	0.61
3C-34	164.54	1566.92	2489.13	0.63
4C-36	109.52	(896.97)[2]	1629.06	-
4C-35	136.98	1312.18	1742.45	0.75
4C-34	130.36	(1419.98)[3]	1715.67	0.83

	Shear Demand(Kip)	Shear Capacity(Kip)	Demand /Capacity
1C-36	36.50	38.90	0.94
1C-35	41.07	39.45	1.04
1C-34	36.58	39.26	0.93
2C-36	(12.34)[1]	32.73	-
2C-35	35.88	33.70	1.06
2C-34	28.86	33.49	0.86
3C-36	21.44	26.99	0.79
3C-35	32.75	27.89	1.17
3C-34	28.17	27.71	1.02
4C-36	(15.55)[2]	25.60	-
4C-35	23.94	26.65	0.90
4C-34	(24.46)[3]	26.40	0.93

1. 2FSB1 yielded
2. 4FSB1 yielded
3. 4FSB3 yielded



6. Joint demand

for $2nd-7th$ *floor*

$$exterior\ \ joint: V_{demand} = T_{beam} - V_{col}\ \ ,\ \ T_{beam} = A_s f_y\ \ and\ V_{col} = \frac{M_b}{(h_{below} + h_{above})/2}$$

$$interior\ \ joint: V_{demand} = T_{beam,left} + T_{beam,right} - V_{col}\ \ ,\ V_{col} = \frac{\max\left[M_{b1,negative} + M_{b2,positive})\right]}{(h_{below} + h_{above})/2}$$

for roof $joint: V_{demand} = T_{beam},\ T_{beam} = A_s f_y$

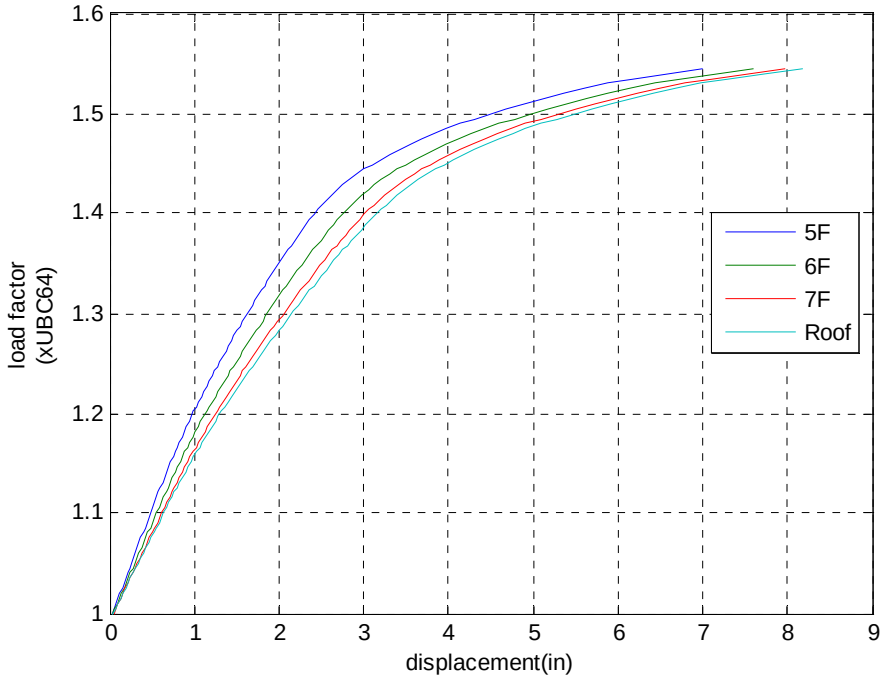
Joint Shear	Demand Capacity(ACI)		Demand Capacity(ACI)		Demand Capacity(ACI)
2J9	65.11(63.78)	2J8	128.57(125.99)	2J7	93.13(91.73)
	162		203		203
5J9	66.91(65.08)	5J8	119.15(116.09)	5J7	108.25(105.70)
	140		175		175
RJ9	48	RJ8	111.2	RJ7	98.4
	94		140		140

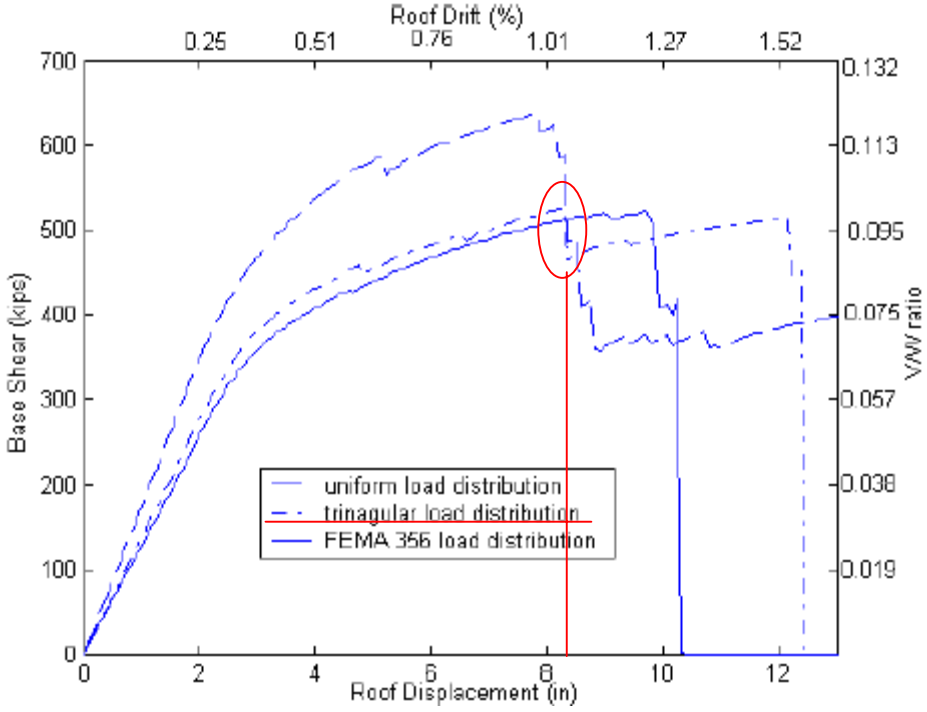
-.Used Moment Capacity

	Negative Moment Capacity(Kip-in)		Positive Moment Capacity(Kip-in)	
	Interior	Exterior	Interior	Exterior
2FSB-1	-2301.6 (2524.4)	-1964.9 (2141.2)	1585.6 (1701.3)	1584.4 (1701.3)
2FSB-2	-1962.1 (2141.2)	-2299.6 (2524.4)	951.49 (956.6)	951.5 (956.6)
2FSB-3	-1962.1 (2141.2)		951.49 (956.6)	
FSB-1(5F)	-1559.7 (1785.9)	-1335.3 (1521.6)	852.53 (938.8)	851.64 (938.8)
FSB-2(5F)	-1559.7 (1785.9)	-1559.7 (1785.9)	658.89 (692.6)	658.89 (692.6)
FSB-3(5F)	-1559.7 (1785.9)		658.89 (692.6)	
RFSB-1	-1048.1 (1183.5)	-635.55 (671.2)	826.24 (914.8)	821.42 (914.8)
RFSB-2	-1046.5 (1183.5)	-1046.5 (1183.5)	638.30 (675.0)	638.30 (675.0)
RFSB-3	-1046.5 (1183.5)		638.30 (675.0)	

7. Push-over Analysis Results (Load Control)

-. Roof displacement vs applied Load Factor(multiply by UBC Lateral Force)





[Chaitanya Paspuleti, MSCE Dissertation, University of Washington(2002)]

-. Failure sequence from analysis results and comparison of demand-capacity ratios

(1) 2nd floor beams flexural failure

(2) 4th and 5th story beams flexural failure

(3) 1st ,2nd and 3rd floor column shear failure

-. Approximate the relation between 2nd floor beam Shear vs 1st Column Axial force

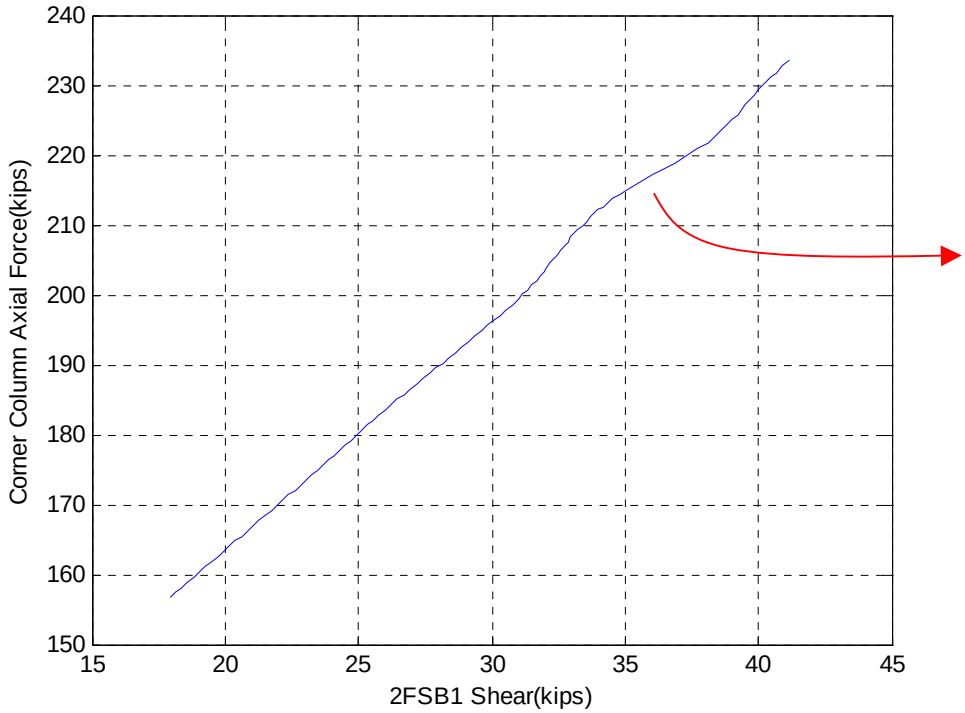
Column dead loads of Van Nuys Database.

Gravity loads - DL + 10 PSF partition load

Floor	column	
	Exterior	Corner
7	29.8	20.2
6	63.8	43.2
5	97.8	66.2
4	131.8	89.2
3	165.8	112.2
2	199.8	135.2
1	237.3	161.1

Column dead loads + axial loads due to overturning

Floor	Exterior		Corner
	C34	C35	C36
4	130.4	136.3	112.2
3	164.6	168.3	149.5
2	198.3	202.0	185.2
1	235.5	237.4	233.6



Linear regression

$Y = 3.3X + 97$

X ; Beam shear

Y : Axial Force

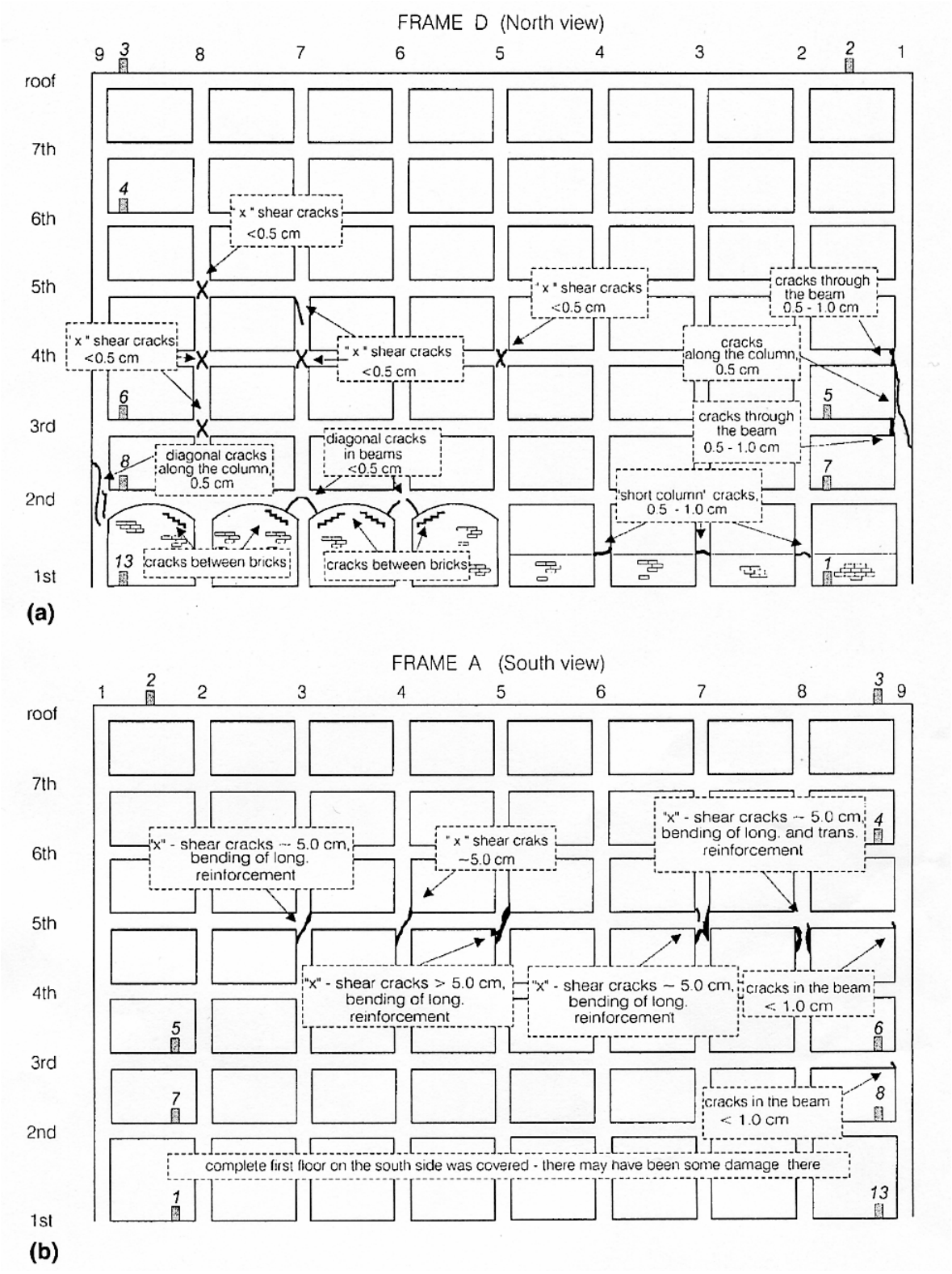


Figure 2.7 Observed Damage Pattern after the Northridge Earthquake (Trifunac, 2001)

8. Alternative of Test Setup

-. Red line : 6’ beam(half span), Purple line : 10’ beam(half span), Green line : 12’ beam(half span)

-. Basic frame is supported two(or one) diagonal braces as the left figure

-. Not real scale members