SEAOC Seismic Design Manual (1997 UBC Version)
Errata No. 2 for Volume III

Page 22, Design Example 1A.

After "Roof Weights" add the following comment:

> Where the top story partitions are lateral braced at the roof, it may be appropriate to include a 5 psf partition weight in the roof dead loads for determination of seismic forces.

Page 33, Design Example 1A.

Line 12, change to read:

> $T_A = C_t\ (h_n)^{3/4}$, where $C_t = 0.020$

Page 43, Design Example 1A.

Line 1, change to read:

> The computer model assumes rigid diaphragms <u>for</u> load distribution to the frames.

Page 45, Design Example 1A.

Line 14, change to read:

> With allowable diaphragm shear of 1.75 k/ft. . .

Line 20, change to read:

> The force at Line 1 is:

Page 47, Design Example 1A.

After Line 8, add the following paragraph:

> **Comment:**
> The use of slab reinforcing in the concrete fill over metal deck for the seismic collector as shown above represents a "standard of practice" among some structural engineers. As noted at the bottom of page 46, an alternate technique is to use the wide flange beams and connections for the seismic tie. Some jurisdictions have interpreted §1921.6.7.3. (which places limits on spacing and concrete cover for collector reinforcing in concrete slabs) as applying to concrete fill over metal deck. Under the provisions of this section, smaller diameter bars

would be required in the 3¼" concrete fill to meet the cover requirement and transverse confinement may be required pending analysis of compression in the slab "boundary element" per §1921.6.2.3. The design shown in this example, with the designated collector placed as reinforcing in the concrete fill and with wide flange beams supporting vertical load, has considerable redundancy because the wide flange beams also can act as a collector. This situation is markedly different from the type of concrete parking structure construction where chords are placed in the topping slab and there is no edge beam to provide redundancy. Failures of this system in the 1994 Northridge earthquake lead to the provisions of §1921.6.7.3.

Page 58, Design Example 1A.

Line 1, change equation to:

$R_{BS} = [0.3(36) + 0.5(8)](58)(t_{min}) = \underline{800.4}$ kips

Page 72, Design Example 1B.

Add the following sentence to end of second paragraph:

> The two story "X-bracing" configuration is also retained, obviating the 50 percent force increase required for brace members in chevron configurations per §2213.8.4.1.

Page 94, Design Example 2.

Line 6, change "Δ" to "ρ"

Line 9, change "Δ" to "ρ"

Line 16, change to:

$$A_B = 212' \times 152' = 32{,}224\ ft^2$$

Page 95, Design Example 2.

Table 2-1, Building mass calculation, should appear as follows:

Level	Floor Area (s.f.)	w_i (psf)	$W_{r/f}$ (kips)	Length exterior walls (ft)	h walls (ft)	w_i walls (psf)	W walls (kips)	W_i (kips)
Roof	32,224	74	2,385	728	10	20	146	2,530
5	32,224	76	2,449	728	12	20	175	2,624
4	32,224	76	2,449	728	12	20	175	2,624
3	32,224	76	2,449	728	12	20	175	2,624
2	32,224	76	2,449	728	13	20	189	2,638
Totals	161,120		12,181				860	13,040

Page 95, Design Example 2.

In Part 3a, change expression for design base shear to read:

$$V = 1.13 \times 0.189W = 1.13 \times 0.189(13,062) = \underline{\underline{2,789k}}$$

(Note: the originally published base shear of 13,062 k is used for the remainder of the example instead of 13,040 k.)

Page 96, Design Example 2.

Table 2-2, Vertical distribution of shear, should appear as follows:

Level	w_x (k)	w (k)	h_x (ft)	h (ft)	$w_x h_x$ (k-ft)	$\frac{w_x h_x}{\Sigma w_i h_i}$ (%)	F_x (k)	ΣV_i (k)
R	2,530		62		156,871	32%	887	
		2,530		12				887
5	2,624		50		131,187	27%	742	
		5,154		12				1,629
4	2,624		38		99,702	20%	564	
		7,778		12				2,193
3	2,624		26		68,217	14%	386	
		10,401		12				2,598
2	2,660		14		37,242	7%	211	
		13,062		14				2,789
Totals	13,062				493,220	100%	2,789	

Page 113, Design Example 2.

Change second paragraph to read:

> Using plastic design procedures outlined in AISC Section N, obtaining forces from a computer analysis, and showing calculations in tabular form, design forces for braces (P and M) are calculated as <u>1.3Ω</u> times seismic forces plus 1.3 times gravity forces. Column shear forces are not a controlling factor and are. . . . . .

Change Table 2-8a, Brace forces, to read:

LEVEL	P_E E/1.4	M_E E/1.4	Ω	Brace Overstress Factor	P_D D	M_D D	Brace Overstress Factor	P Design	M Design
5	106	10.2	3.16	<u>1.3</u>	11.8	5.1	<u>1.3</u>	519.5	55.9
4	194	11.7	2.02	<u>1.3</u>	14.6	4.4	<u>1.3</u>	609.3	42.0
3	262	23.4	1.69	<u>1.3</u>	14.7	4.3	<u>1.3</u>	686.0	65.7
2	302	26.7	1.44	<u>1.3</u>	14.4	4.3	<u>1.3</u>	672.4	64.0
1	372	38.5	1.62	<u>1.3</u>	13.9	3.4	<u>1.3</u>	927.2	98.9

Page 114, Design Example 2.

Change Table 2-9a, Design column forces, to read:

LEVEL	P_E E/1.4	M_E E/1.4	Ω	Column Overstress Factor	P_D D	M_D D	Column Overstress Factor	P Design	M Design
5	106	10.2	3.16	1.25	11.8	5.1	1.25	432.9	46.6
4	194	11.7	2.02	1.25	14.6	4.4	1.25	507.7	35.0
3	262	23.4	1.69	1.25	14.7	4.3	1.25	571.7	54.8
2	302	26.7	1.44	1.25	14.4	4.3	1.25	560.3	53.3
1	372	38.5	1.62	1.25	13.9	3.4	1.25	772.6	82.4

Page 121, Design Example 2.

Line 15, change "$2V_p/e$" to "$2M_p/e$".

Page 136, Design Example 2.

Change Table 2-15, Final frame member sizes for EBF4 (LRFD), to read:

Level	Beams	Links	Beam Cover Plate (in.) (1)	Columns	Braces
Roof	W14x38	32"	6” x ¼"		
5	W16x89	48"	6” x ¼"	W12X87	W12X87
4	W21x111	56"	6” x ¼"	W12X87	W12X152
3	W21x122	56"	6” x ¼"	W12X87	W12X210
2	W27x178	66"	Not req’d	W12X170	W12X230
1				W12X170	W12X252

Note:
1. Top and bottom flanges outside link.

Page 145, Design Example 3A.

After “Roof Weights” add the following comment:

> Where the top story partitions are laterally braced at the roof, it may be appropriate to include a 5 psf partition weight in the roof dead loads for determination of seismic forces.

Page 208, Design Example 3B.

Lines 11 and 12, change to read:

> See Design Example 1A, Part 6g for a beam-to-column shear plate connection design.

Page 273, Design Example 6.

Revise text as follows:

> *Partitions are 20 psf for gravity calculations and 10 psf for seismic calculations.

Page 295, Design Example 6.

Revise Line 17 as follows:

$$1.1\,(1.2D + 0.5L + 1.0E + 0.22D) = 1.58D + 0.55L + \underline{1.1}E_h \qquad (12\text{-}5)$$

Page 300, Design Example 6.

Revise expressions at bottom of page as follows:

$$\sum M_{e,interior} = \frac{2(2{,}550 kip - ft.)}{0.7} = 7{,}284 kip - ft \geq \frac{3{,}882 kip - ft}{0.9} = 4{,}143 kip - ft$$

$$M_{e,end} = \frac{2(2{,}450 kip - ft.)}{0.7} = 7{,}000 kip - ft \geq \frac{2{,}275 kip - ft}{0.9} = 2{,}527 kip - ft$$