APPENDIX C: AMMENDMENTS AND CLARIFICATIONS TO ASCE 41

1.0 Introduction

ASCE 41-13 is the seismic evaluation and retrofit design standard that is currently approved under Ordinance 17-1011 for Non-Ductile Concrete and Pre-Northridge Steel Moment Frames. This standard is updated every few years to incorporate new evaluation techniques and research findings as well as to improve the overall quality/clarity of the document. Currently ASCE 41-17 has already been published. This appendix incorporates some of the updates that are found in ASCE 41-17 as well as clarifying the use of some technical requirements when using the standard to meet the intent of the ordinance.

2.0 Amendments and Clarifications

The following amendments and clarifications shall supplement the requirements of ASCE 41-13.

1. **Building Separation:**

   For the purposes of satisfying the requirements of ASCE 41 Section 7.2.13 for the targeted building, an approved analysis procedure that accounts for the change in dynamic response of the structures caused by impact may be evaluated analytically or qualitatively. This evaluation/retrofit will require a structural peer review (see Appendix C: Peer Review Requirements section 2.1).

2. **Geologic Site Hazards:**

   Geologic hazards shall be evaluated/mitigated per ASCE 41 Section 8.3 for collapse potential under the BSE-2E Hazard. This evaluation/retrofit will require a structural and geotechnical peer review (see Appendix C: Peer Review Requirements sections 2.1 and 2.2).

3. **Detailing of new elements:**
   a. Independent new component: Must meet Ordinary detailing requirements from respective material standard.
   b. Composite component: Must meet Ordinary detailing where possible, however where it is not possible to meet full detailing requirements, design professional should submit analysis approach as a Basis of Design for city approval. The city may trigger add-ons for part of a Design Criteria depending on the complexity of the element.

4. **Nonlinear Static Pushover Displacement Demand:** Although component Acceptance Criteria (AC) need not be evaluated at roof displacements beyond the Target Displacement, δ_t, pushover displacements shall be required to show global stability at 150% of the Target Displacement as per ASCE 41-13 Section 7.4.3.2.1. ASCE 41-13 Nonlinear Dynamic Procedure (NDP) shall be required where global stability cannot be demonstrated at this displacement demand. It shall be permitted to allow exceedance of local element modeling and deformation limits for the purpose of this check.

5. **Energy Balance for Nonlinear Analyses:** The calculated energy balance for nonlinear analyses shall not exceed 5% unless it can be shown there are no errors in equilibrium and there is insignificant sensitivity to the chosen energy balance ratio.

6. **Steel Column Splice Acceptance Criteria:** The lower-bound tensile strength of the partial-joint-penetration groove welded splices shall be determined in accordance with Equation (7-1) below. Demand on the splice shall be determined as the maximum stress in the smaller section at the tip of the partial-joint-penetration groove weld or in accordance with Equation (7-2).

$$\sigma_{cr} = \frac{K_{IC}}{F\left(\frac{a_0}{t_{f,u}}\right)\sqrt{\pi a_0}} \le F_{ue}\left(1 - \frac{a_0}{t_{f,u}}\right) \le F_{ye} \qquad (7\text{-}1)$$

where:

$$F\left(\frac{a_0}{t_{f,u}}\right)=\left(2.3-1.6\frac{a_0}{t_{f,u}}\right)\left(4.6\frac{a_0}{t_{f,u}}\right)$$

K_{IC} = fracture toughness parameter, per Table 7-1, ksi√in (MPa√mm). If the Charpy v-notch toughness is not known, it is permitted to use the value for 10 ft-lb.

a_0 = dimension of the smaller flange or web thickness that is not welded, in. (mm)
$t_{f,u}$ = thickness of the flange or web, in. (mm)

$$\sigma_{UF}=\left(\frac{P_{UF}}{A_g}\right)\pm\left(\frac{M_{UF,x}}{S_x}\right)\pm\left(\frac{M_{UF,y}}{S_y}\right) \tag{7-2}$$

where
A_g = Gross area of the smaller member, in.^{2} (mm^2)
S_x = Elastic section modulus of the smaller member taken about the x-axis, in.^{3} (mm^3)
S_y = Elastic section modulus of the smaller member taken about the y-axis, in.^{3} (mm^3)

TABLE 7-1
Fracture Toughness Parameters

Charpy V-Notch at LAST, ft-lb (J)	K_{IC}, ksi$\sqrt{\text{in.}}$ (MPa$\sqrt{\text{mm}}$)
5 (6.8)	50 (1740)
10 (14)	100 (3470)
20 (27)	185 (6430)
40 (54)	300 (10 400)

7. **Panel Zone Acceptance Criteria:** When using nonlinear procedures, where V_{pz} / V_y > 1.10, determined in accordance with Section 9.4.2.4.3, Item 4.2, and the beam-flange-to-column-flange connection is made with notch-tough complete joint penetration (CJP) welds that satisfy the requirements of AISC 341 and the welds are located at the edge of the panel zone where column flanges are susceptible to kinking, the permissible plastic rotation angle of the panel zone for the LS and CP Performance Levels shall not exceed the limit determined in accordance with Equation (7-3).

$$\frac{0.183F_y}{G}\left(\alpha+\frac{3.45}{\alpha}\right)\left[1-\left(\frac{|P|}{2P_{ye,cf}}\right)^2\right]\le\gamma_{p,pz} \qquad (7\text{-}3)$$

where

$\alpha = d_b/t_{cf}$;

d_b = Smallest depth of the connecting beams;

t_{cf} = Thickness of the column flange;

$P_{ye,cf}$ = Expected axial yield capacity of the column flange, = $A_{cf} \times F_{ye}$; and

$\gamma_{p,pz}$ = Plastic rotation angle (plastic shear strain) computed from Table 7-2.

Otherwise, where the beam-flange-to-column-flange connection is made with CJP welds that do not satisfy the requirements of AISC 341, the permissible plastic rotation angle of the panel zone for the LS and CP Performance Levels shall not exceed the limit determined in accordance with Equation 7-4.

$$\frac{0.092F_y}{G}\left(\alpha+\frac{3.45}{\alpha}\right)\left[1-\left(\frac{|P|}{2P_{ye,cf}}\right)^2\right]\le 0.5\gamma_{p,pz} \qquad (7\text{-}4)$$

TABLE 7-2

	Modeling Parameters			Acceptance Criteria		
	Plastic Rotation Angle, Radians		Residual Strength Ratio	Plastic Rotation Angle, Radians		
Component or Action	***a***	***b***	***c***	**IO**	**LS**	**CP**
Column Panel Zones—*Shear*						
For $\lvert P\rvert/P_{ye} < 0.4$	$12\gamma_y$	$12\gamma_y$	1.0	$1\gamma_y$	$12\gamma_y$	$12\gamma_y$
For $\lvert P\rvert/P_{ye} \ge 0.4$	$20(1-\lvert P\rvert/P_{ye})\gamma_y$	$20(1-\lvert P\rvert/P_{ye})\gamma_y$	$5/3(1-\lvert P\rvert/P_{ye})$	$5/3(1-\lvert P\rvert/P_{ye})\gamma_y$	$20(1-\lvert P\rvert/P_{ye})\gamma_y$	$20(1-\lvert P\rvert/P_{ye})\gamma_y$

8. **Concrete Inadequate Lap Splice Modeling:** Lap splices in concrete elements shall be checked in accordance with Section 10.3.5. Where inadequate lap splices are encountered, component strength shall be reduced

per Section 10.3.5, while modeling parameters (MP) and deformation acceptance criteria (AC) for inadequate lap splice regions shall be per their respective tables in Chapter 10 where tables include information for inadequate lap splice conditions. Where inadequate lap splice information is not included for the respective component MP and AC table, AC shall be per the component AC tables with respect to all other detailing conditions and demands, and nonlinear modeling of lap splices shall be modeled per section 10.3.5 in nonlinear procedures, for example in lap splices of vertical concrete wall reinforcement. For linear procedures, the permissible “m” values for LS and CP shall not exceed 1 and 2, respectively.

9. **Concrete Structural Wall and Coupling Beam Element Modeling:** Structural wall and coupling beam elements shall have MP and AC in accordance with Sections 10.7.2.2 & 10.7.2.4 regardless of the chosen modeling approach. As such, when fiber elements are used to represent nonlinear flexural actions, wall modeling and AC shall be calibrated to demonstrate plastic hinge deformations at strength degradation and strength loss consistent with the respective MP and AC tables. Global wall behavior is highly sensitive to analytical meshing, and thus mesh criteria shall be established to maintain analytical behavior consistent with MP and AC tables.

10. **Axial Demands on Concrete Walls and Columns:** For concrete columns or walls under combined axial load and biaxial bending, the combined strength shall be evaluated considering biaxial bending. When using linear procedures, the axial load P_{UF} or P_{UD} shall be calculated as a force-controlled action or deformation-controlled actions per Section 7.5.2. The design moments M_{UD} should be calculated about each of two orthogonal axes. Combined strength shall be based on principles of mechanics with applied bending moments calculated as $M_{UDx}/(m_x\kappa)$ and $M_{UDy}/(m_y\kappa)$ about the *x*- and *y*-axes, respectively. Acceptance shall be based on the applied bending moments lying within the expected strength envelope calculated at an axial load level of P_{UF} if the member is in compression or $P_{UD}/[(\text{minimum of } m_x \text{ and } m_y)\kappa]$ if the member is in tension.

11. **Concrete Shear Capacity:** Reductions to shear capacity shall be taken per Section 10.3.4, however both concrete and steel components to shear strength (V_c and V_s, respectively) shall be considered effective for shear strength. No reduction to Vc shall be taken as in Chapter 18 of ACI 318.

12. **Shear Transfer for Shotcrete Wall Retrofit:** All force transfer from diaphragms shall be detailed and shown analytically to transfer to existing and new shotcrete components of retrofitted shotcrete walls as required. Sufficient dowel transfer shall be provided between the existing and shotcrete wall for composite action, or the shotcrete wall shall be shown to have sufficient thickness and detailing to act as an independent concrete wall.